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FLAT PLATE ON COLUMNS ELONGATED IN PLAN

by



ARTHUR EDWARD SMITH

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF
MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

SPRING, 1969

UNIVERSITY OF ALBERTA
FACULTY OF GRADUATE STUDIES

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled FLAT PLATE ON COLUMNS ELONGATED IN PLAN submitted by Arthur Edward Smith in partial fulfilment of the requirements for the degree of Master of Science.

ABSTRACT

Theoretical studies on a typical interior panel indicated predominantly one-way action for square panels supported on columns measuring 0.4 of the span length in one direction. In order to observe the behavior of an actual structure, such a slab system was designed by the elastic frame analysis method. The resulting 1/3 scale structure contained nine panels in a three by three grid.

The structure was instrumented in hopes of determining bending moments in the slab and their distribution. Loading was simulated uniform load in various patterns and to various levels.

Due to malfunctioning recording equipment, strain readings were lost. As a result, the analysis was one of general behavior with considerations being given to deflections, crack patterns and ultimate capacity of the slab.

The slab behaved predominantly as if it were a one-way slab spanning perpendicular to the long faces of the columns. General behavior of the structure was satisfactory with regards to deflections and cracking. Tests on the interior panel indicated an overload factor based on deflection limitations of about 1.46.



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ACKNOWLEDGEMENTS

The author wishes to express his sincere appreciation to Dr. S. H. Simmonds, Associate Professor of Civil Engineering, for his guidance in the preparation of this thesis.

Laboratory work was carried out with the expert assistance of Mr. H. Panse and L. Burden. This assistance is greatly appreciated.

The author would also like to express his thanks to Errol Nash for his speed and efficiency in drafting the plates for this thesis, and to his wife, Leslie, for her typing assistance.

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CHAPTER I

INTRODUCTION

1.1 Introductory Remarks.

The design methods for flat slab and flat plate floors were developed through empirical rather than mathematical means. These methods are based on loads being carried by two-way slab action to rather regularly spaced square or circular columns and hence the design procedures are strictly applicable only to this support system. Recent economic and esthetic demands in multistory construction have resulted in variations in the flat plate support systems which are outside the scope of supports originally envisioned. One such variation is the elongation of the cross-section of the supporting column .

1.2 Present State of Art.

Today in construction of multistory apartment and office buildings, the use of columns having cross-sections elongated in one direction only is becoming more and more popular. Many buildings are being constructed with columns which in reality are walls separating rooms and may have a ratio of the long dimension of the column to the span length (c/l) of as high as 0.8.

Although the present American Concrete Institute Building Code (ACI 318-63) (1) does not specifically state that the empirical design method presently in use is not applicable, the forthcoming ACI code does disallow its use for c/l greater than 0.35. Thus the method

of elastic frame analysis must be used.

A previous study on this problem of behavior of such structures by Kavanagh (2) made use of numerical analysis of a typical interior panel only. This study indicates two important conclusions. First, the use of slightly elongated columns in conjunction with flat-plate floors will result in greatly reduced deflections. Secondly, a distinct change in behavior occurs when the c/l value approaches 0.4. At this point the general behavior ceases to be two-way and becomes one-way action perpendicular to the long dimension of the column.

1.3 Object.

The object of this study was to examine the behavior of a slab supported on columns having elongated cross-sections which was designed according to the present ACI code provisions, using the elastic frame analysis.

The column cross-section was considered to be the important variable in such a study. The column size to span length in one direction was chosen as $1/24$, a representative value for this type of construction. As a result of Kavanagh's work, a column size to span length ratio of 0.4 was chosen for the other direction, as this is the approximate value at which the transition from two-way to one-way action occurs.

Of main concern in the test were the deflections, crack patterns and ultimate capacity of the slab. Attempts were also made at the determination of the slab bending moments, their distribution, and the total static moment (M_o) in both directions.

1.4 Scope.

This study involved the testing of a one-third scale model flat plate floor slab, supported on columns elongated in one direction to 0.4 of the span length. The details of design and construction are presented in Chapter II, and the loading apparatus and instrumentation are described in Chapter III.

Observations of the behavior of the test slab were made with various patterns of loaded panels at the following load levels:

1) low loads below the design load, 2) loads up to and slightly beyond the design loads, 3) low loads below the design loads in various patterns, and 4) loading to failure. The results and analysis of these tests are presented in Chapter V in the form of crack patterns, load-deflection plots and observations on general behavior.

Chapter VI compares the observed deflections of the center panel with the theoretical value as computed by Kavanagh and also other common procedures.

Chapter VII contains the summary and conclusions.

CHAPTER II

DESCRIPTION OF TEST SLAB

2.1 Selection of Parameters.

The test structure was designed as a one-third scale of a typical full-size prototype structure. Dimensions were governed by restrictions imposed by the requirements of the test program.

The prototype structure consisted of nine panels, each eighteen feet square and arranged in a three by three grid, as shown in FIGURE 2.1. No edge beams were used.

2.2 Analysis.

The design of the prototype structure was based on the elastic frame analysis in accordance with the 1963 ACI building code for reinforced concrete. The design assumed a live load of 50 psf., a concrete strength, f'_c , of 3000 psi, and an allowable steel stress, f_s , of 20,000 psi.

The slab was assumed to be 6" thick which resulted in a dead load of 75 psf. and a total design load of 125 psf.

The column reactions were assumed to be concentrated at the column centerlines and the total moments were computed on the basis of the full span. These moments were reduced according to the size of supports to give the moments at the column faces.

The columns were assumed to be 9" thick in one direction and 0.4 of the span length (7.2') in the other. The longitudinal reinforcement was designed arbitrarily at 3% of the cross-sectional area.

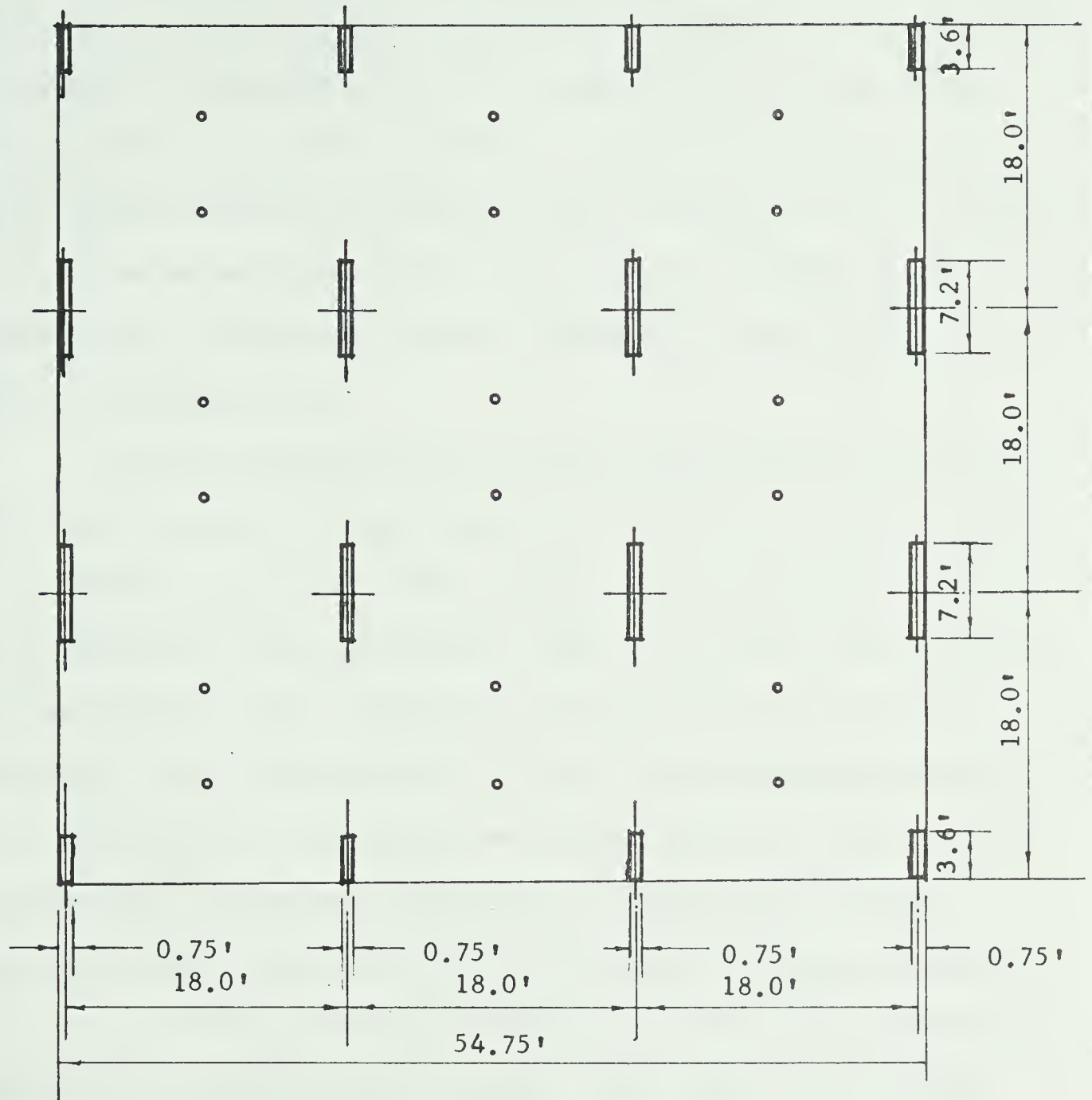


FIGURE 2.1 PROTOTYPE SLAB LAYOUT

2.3 Description of Model Slab.

In the reduction of scale from the prototype to the one-third scale model tested, geometric similarity was maintained and all linear dimensions of the model were $1/3$ of those of the prototype. The applied loads on the model required no scaling since the magnitudes of the uniform load are the same for both the model and the prototype.

The layout of the model slab is shown in FIGURE 2.2 and a general view of the top is shown in FIGURE 2.3. The slab had a nominal thickness of 2".

The slab was supported on columns which extended effectively only below the slab. Column stubs were formed on the top to stiffen the slab over the columns. This was done in an attempt to minimize the curvature of the slab over the column. The column bases rested on steel balls so that rotation was permitted but translation was prevented. Since the columns in the test slab extended effectively only below the slab, an attempt was made to proportion them in such a manner that they behaved comparably to those in the prototype structure, which extended both above and below, and were continuous with the rest of the building. The desired effects were obtained by making the pin ended columns of such a length that their stiffness against rotation in the midplane of the slab was equal to the stiffness of the prototype column extending a full story height (12') above and below the slab and considered fixed at their far ends.

The main slab reinforcement used was $1/8$ inch square and $1/4$ inch round bars. These were used in an attempt to give a direct reduction in scale from the prototype structure, giving the same number of bars in each strip. This worked well for the $1/8$ inch squares

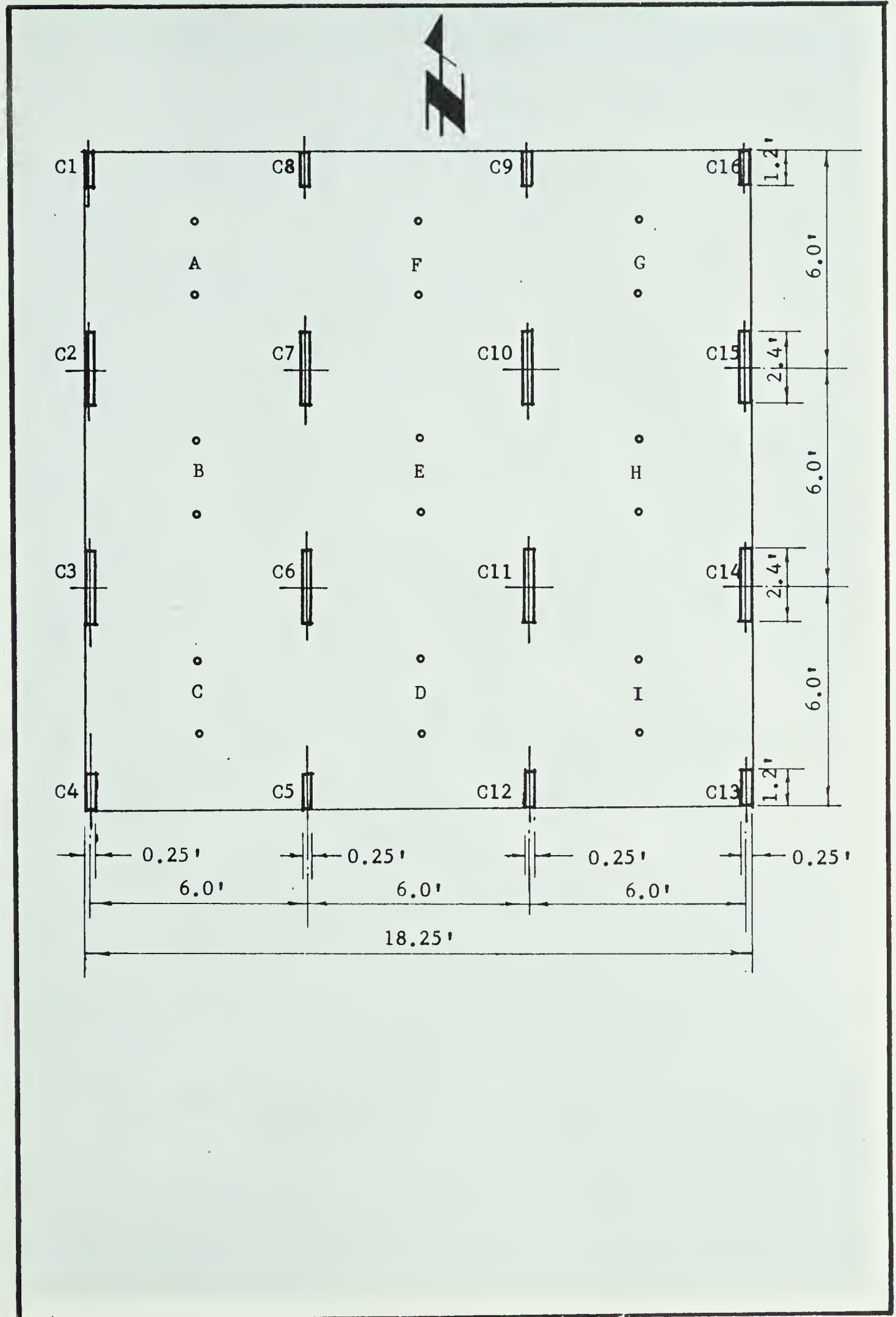


FIGURE 2.2 MODEL SLAB LAYOUT



FIGURE 2.3 GENERAL VIEW OF COMPLETED SLAB

which are almost a direct substitution for the #4 bars in the prototype structure. They did not conform as well in the areas of high moment since the bar spacing would be too close to give practical bar placement. In this region 1/4 inch round bars were used but in no case was the maximum bar spacing of three times the slab thickness exceeded. TABLE 2.1 compares the area of steel required by the elastic design method and the area of steel provided. Details of the slab reinforcing are given in FIGURES 2.4 and 2.5.

It is noted that the area of steel provided is generally equal to that required except where the requirements for minimum steel (temperature and shrinkage) govern the design. This is the case for steel requirements over the interior columns parallel to the long faces of the columns.

The column reinforcement was arbitrarily chosen as 3% and consisted of 3/8 inch deformed bars vertically with 1/8 inch diameter ties at 3 inches on center, as shown in FIGURE 2.6.

For convenience, the panels were designated by letters A to I and the columns were numbered consecutively from C1 to C16. The reference system is also shown in FIGURE 2.2.

2.4 Materials.

The slab reinforcement consisted of 1/8 inch square steel bars and 1/4 inch diameter steel rods. Due to shipping error, the square stock was supplied in two shipments and it was not discovered until after pouring that there existed a considerable difference in properties. The majority of the reinforcing was in the first shipment and had an average yield strength of approximately 45,900 psi and an

TABLE 2.1

COMPARISON OF STEEL AREAS REQUIRED TO THOSE PROVIDED

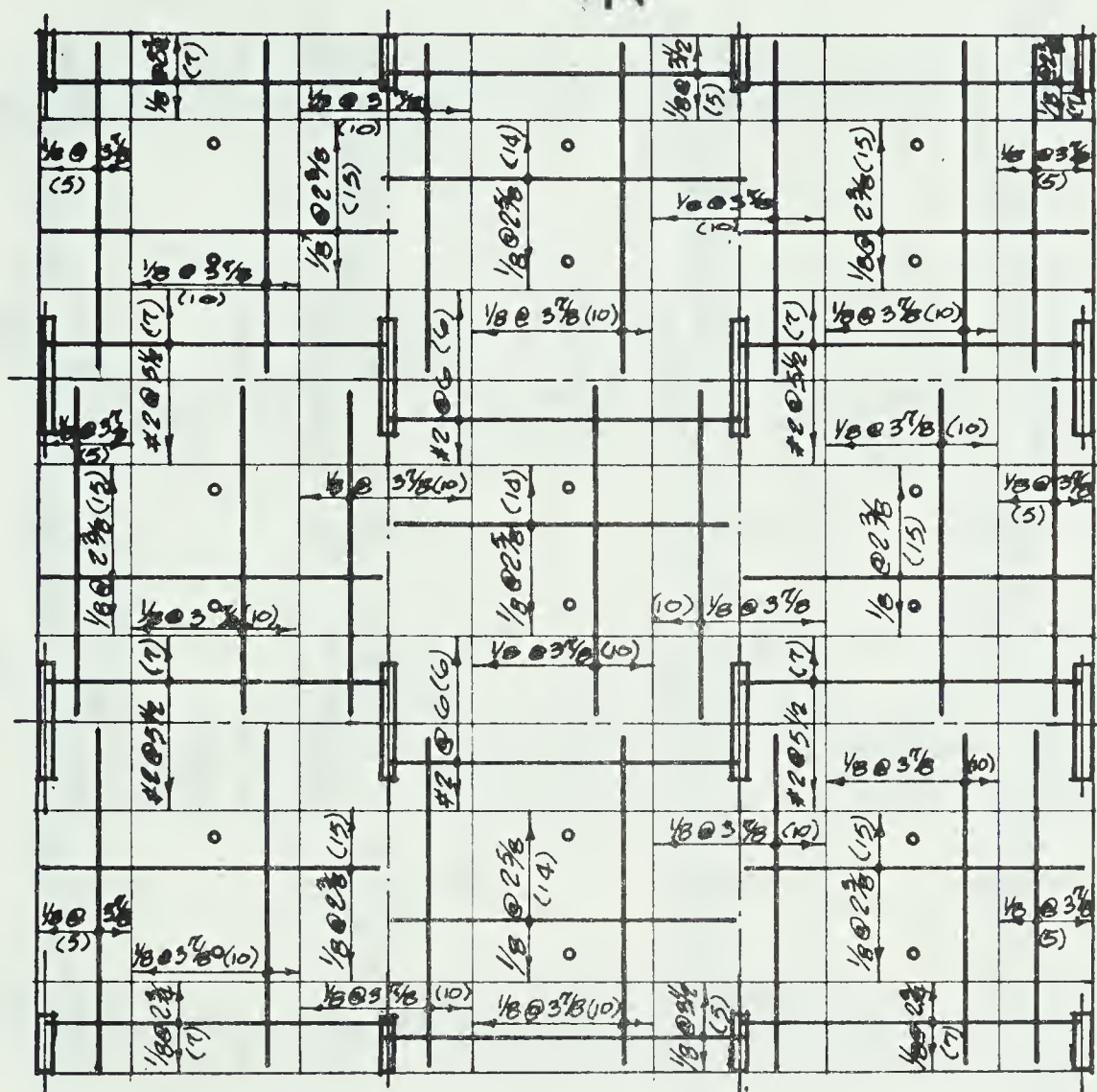
Section				
	1	2	3	4
North-South Line at Center Line of Structure				
Area Required	0.027	0.066	0.099	0.066
Area Provided	0.048*	0.066	0.098	0.066
North-South Line at Interior Supports				
Area Required	0.070	0.079	0.250	0.079
Area Provided	0.071	0.079	0.250	0.079
North-South Line at Center Line of Exterior Panels				
Area Required	0.034	0.071	0.107	0.071
Area Provided	0.048*	0.071	0.108	0.071
North-South Line at Exterior Supports				
Area Required	0.048	0.050	0.199	0.050
Area Provided	0.048	0.050	0.200	0.050
East-West Lines at Interior Supports				
Area Required	0.019	0.019	0.060	0.019
Area Provided	0.048*	0.048*	0.060	0.048*

* Indicates temperature steel governs.

Notes: 1. All areas shown are in terms of square inches per foot.

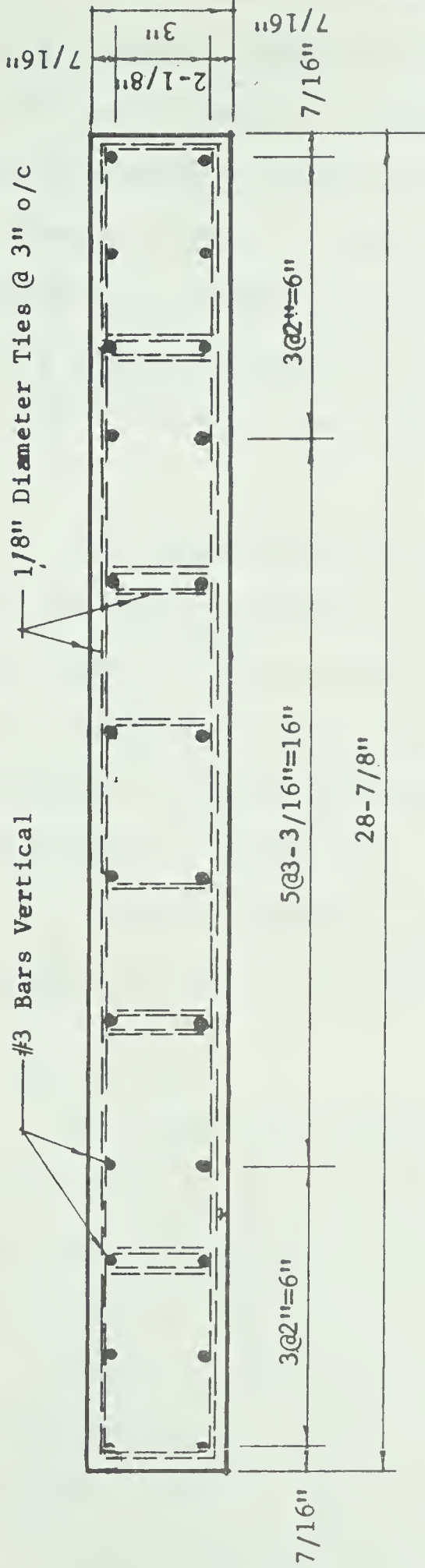
2. Distribution of bending moments in analysis was 70% of negative moment and 60% of positive moment to column strip.

3. Temperature steel governs for all locations on east-west lines except over interior columns.

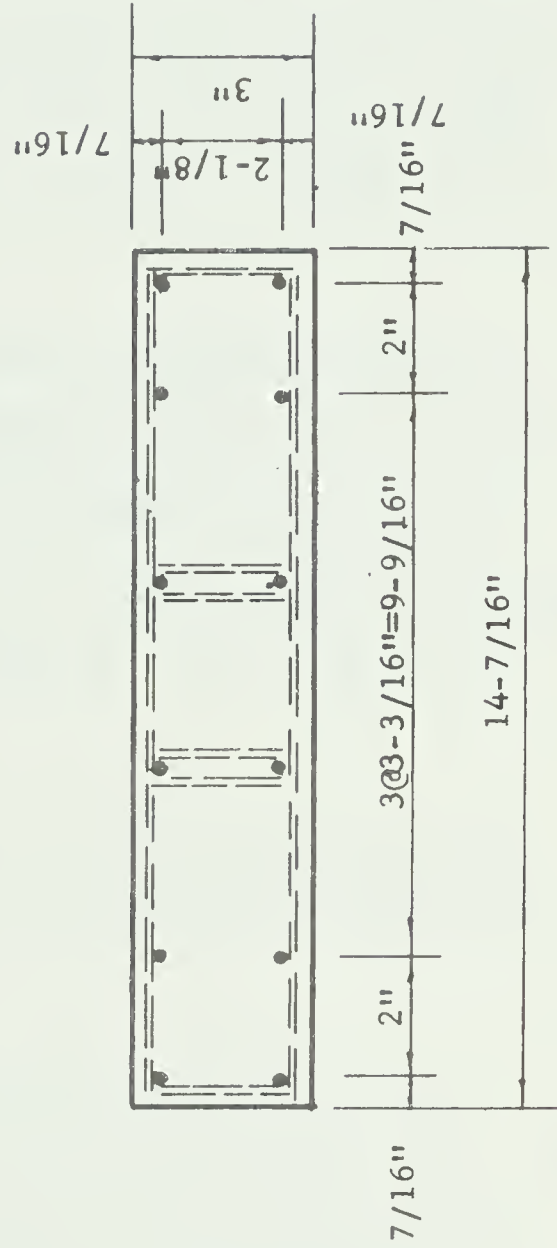


Note: All bottom steel running in direction parallel to the long faces of the columns is minimum temperature steel ($1/8$ inch square @ $3-7/8$ inch).

FIGURE 2.4 TEST SLAB BOTTOM STEEL



Typical Reinforcing in Columns 2, 3, 6, 7, 10, 11, 14, 15



Typical Reinforcing in Columns 1, 4, 5, 8, 9, 12, 13, 16

FIGURE 2.6 COLUMN REINFORCING

average ultimate strength of 64,450 psi. The second shipment had an average yield strength of approximately 70,800 psi and an average ultimate strength of 87,300 psi. The stress-strain curves for both are shown in FIGURE 2.7. The 1/4 inch diameter bars had an average yield point of 43,000 psi and an average ultimate strength of 59,200 psi. A typical stress-strain curve for this rod is shown in FIGURE 2.7. The positions of the various rods are shown in FIGURES 2.4 and 2.5.

The concrete used was ordered from a commercial ready-mix firm. The order specified 3/8 inch maximum aggregate size and Type I normal cement, 3000 psi strength and 4 inch slump. However, as is usually the practice, the company supplied a slightly higher strength than desired, averaging approximately 3500 psi. The stress-strain curve is shown in FIGURE 2.8.

A total of 20 control cylinders were taken during the pour, along with two slump tests, according to the schedule in TABLE 2.2.

2.5 Construction.

The formwork was designed with special attention to the rigidity necessary to control the dimensions of the model. The bottom form was 1/2 inch fir plywood. This form was supported on 2" x 6" joists at 16" on center.

Support for the formwork was provided by the concrete pedestals which finally held the slab at the desired height. Sufficient lateral bracing was provided to avoid a catastrophe.

The side forms for the slab were provided by a 2" x 4" board ripped to 2" to form the side along the west and east edges of the slab and a 2" x 6" board fastened in such a way that the top was 2"

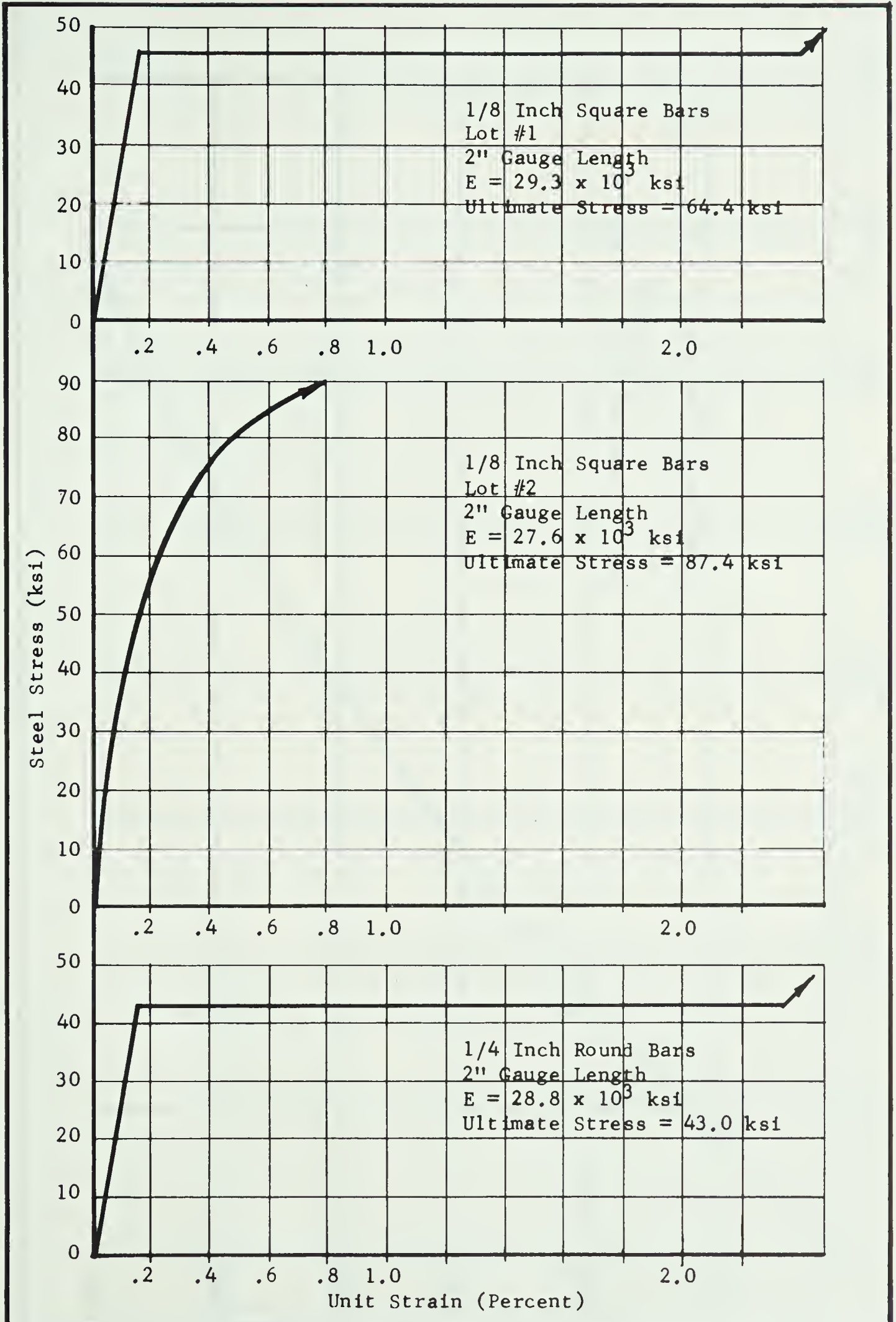


FIGURE 2.7 STRESS-STRAIN CURVES FOR REINFORCING STEEL

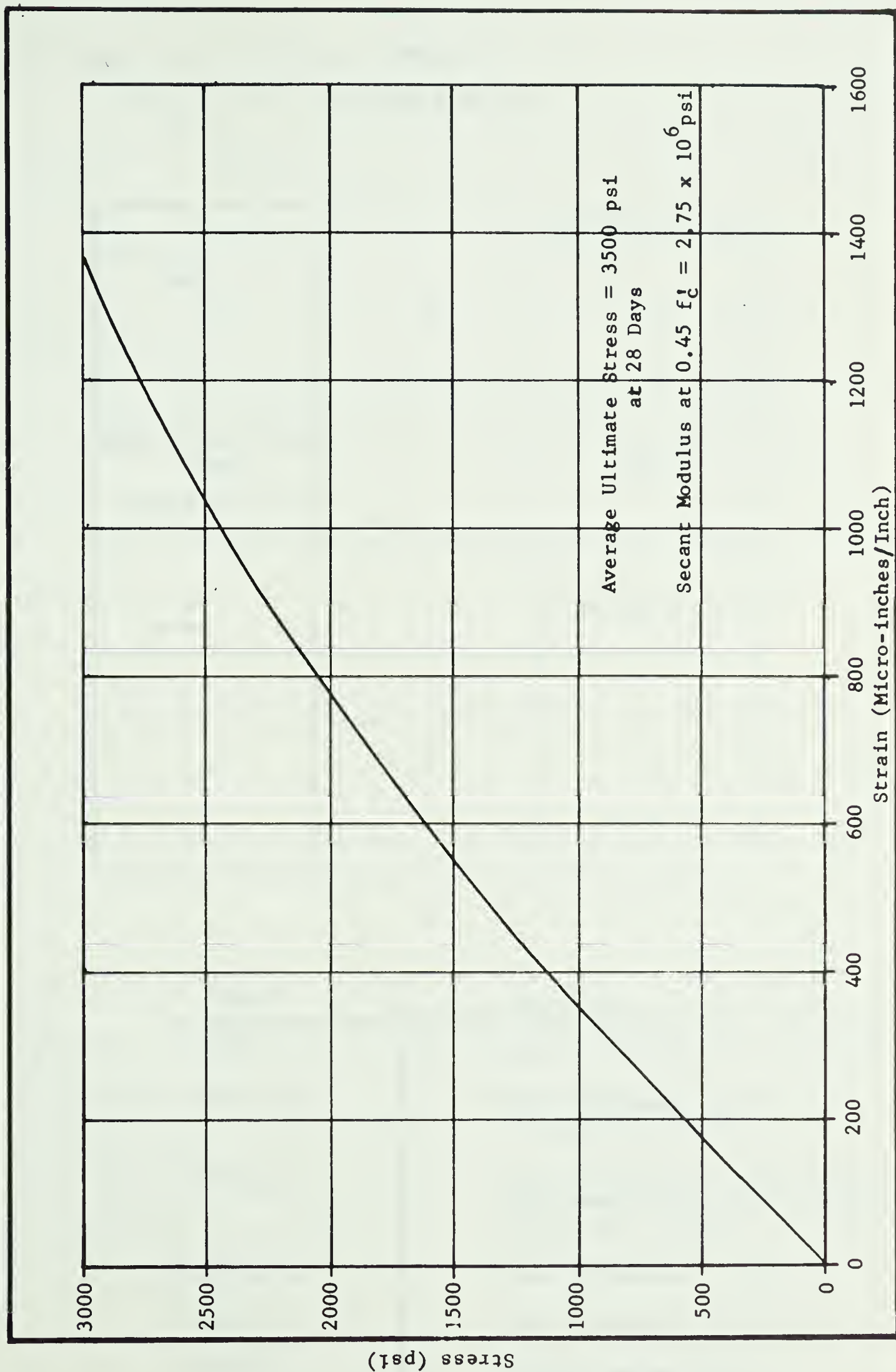


FIGURE 2.8 CONCRETE STRESS-STRAIN CURVE

TABLE 2.2

SLAB POURING LOG

 G	H	Cylinders 9 - 20 I
Moment-Steel Strain Beams Cylinders 5 - 8 F	E	 D
		Cylinders 1 - 4
A	B	C 

Time	Operation
9:00 a.m.	- Begin pouring.
9:45 a.m.	- Cast cylinders 1 - 4. - Slump test, 3-1/2".
10:15 a.m.	- Cast cylinders 5 - 8. - Cast moment-steel strain beams. - Slump test, 2-1/2".
10:45 a.m.	- Cast cylinders 9 - 12.
11:15 a.m.	- Cast cylinders 13 - 20.
11:30 a.m.	- Install column tops. - Finish pouring operation.

above the bottom form, making the north and south edges.

In order to ensure as flat a slab as possible, the formwork was shimmed to approximately the same elevation as determined from a surveyor's level.

Sections of 2" copper tubing were cast into the slab at appropriate locations in order to facilitate installation of the loading apparatus.

All reinforcement was previously tied into mats in order to allow easier placement. The reinforcing was placed beginning with the columns. Following this, the bottom layer of slab reinforcement was laid in place on chairs made from 1/4 inch rods about one inch long. Care was taken in placing the steel in order to keep the steel free from oil.

Finally, the top layer of reinforcing was set into place on chairs fabricated from 1/16 inch diameter welding rod. In order to provide more lateral support to the top layer during pouring, nails were driven randomly into the forms on either side of the reinforcing bars.

The concrete was placed by shovel in order to ensure that the steel positions were not altered. The concrete in the columns was rodded into position since the column was too narrow to get an internal vibrator inside. The concrete in the slab was vibrated into position using an air powered impact hammer on the outside of the formwork. Generally this procedure was satisfactory except for the columns where poor compaction was obtained.

CHAPTER III

INSTRUMENTATION AND LOADING APPARATUS

3.1 Loading Apparatus.

The loading of the model slab was accomplished by means of a system of hydraulic jacks. Each of the nine panels was equipped with its own jack which was connected to a central manifold which enabled, by means of valves, the application of any loading pattern with one pump.

The loading was transmitted from the jacks which were below the testing bed to the load distribution system by means of two high strength $3/4$ inch steel rods per panel, as shown in FIGURE 3.1. Each jack, 10 ton capacity, supplied a concentrated load to the center of a steel load distribution system. This system distributed the load evenly to sixteen points on each panel. At each of these points was another small, wooden distributor which distributed the load to four 2 inch square pads. This, then, broke the load into 64 point loads per panel, as shown in FIGURE 3.2.

The hydraulic pressure was supplied to the manifold by means of an Amsler pump. The jacks were calibrated using a load cell in order to derive a load-pressure dial relationship. This calibration was accurate to within 3 psf.

The load distribution system consisted of a system of H-frames as shown in FIGURE 3.2. Rectangular steel sections were used for all members but the main beam which was made from a 5" wide WF section. All joints in the system were pinned in order to maintain stability,

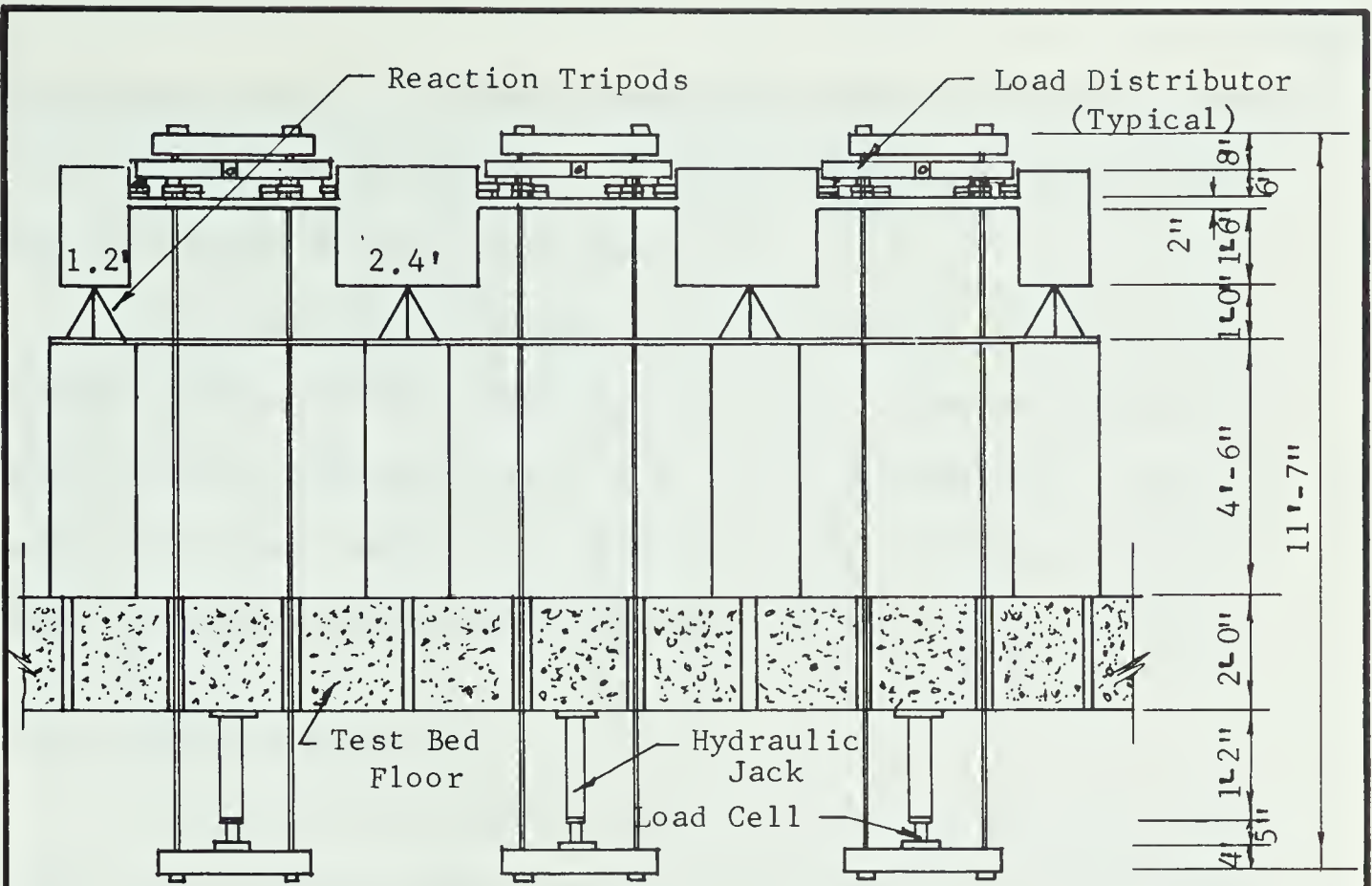


FIGURE 3.1 LOADING APPARATUS

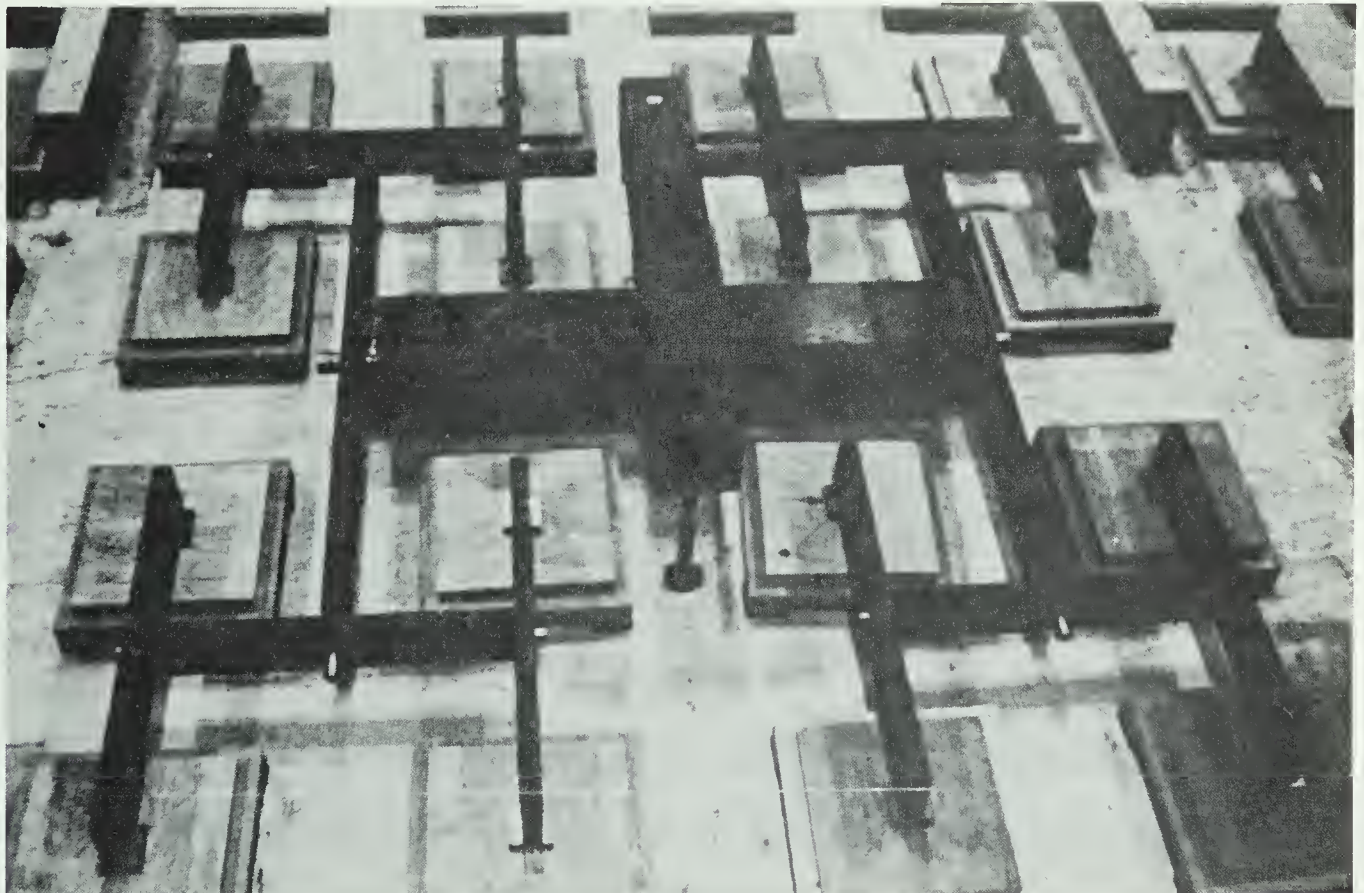


FIGURE 3.2 LOAD DISTRIBUTOR

yet allow rotation. Polished chrome steel balls were used to transmit the load from the tie-bar beams to the 5" WF section and also from the distributor system to the load pads.

The load pads consisted of 11" x 11" x 3/4" plywood plates backed up by a 9" x 9" x 3/4" plywood plate. These were nailed and glued together and rested on 2" x 2" x 3/4" plywood feet. The feet were then covered with a 2" x 2" x 3/8" piece of natural rubber in order to distribute the load uniformly over the concrete surface.

3.2 Measuring Equipment.

Equipment was designed and built to measure the column reactions. The sixteen tripods, one for each column, were instrumented and calibrated to measure the components of each column reaction in three orthogonal directions.

The tripod, shown in FIGURE 3.3, had a triangular steel base plate, 12 inches on a side and 3/4 inch thick. In order to derive the most sensitivity as possible, the leg size was determined on the basis of a maximum stress of about 20,000 to 25,000 psi. In some cases this resulted in impractical sizing and high strength steel was employed which allowed a higher maximum stress.

The tripod was loaded through a 3/4 inch polished chrome steel ball. The design was carried out in such a manner that the axes of the legs intersected at the center of the ball.

The legs were instrumented with two type A-7 SR-4 strain gauges on each leg. The gauges were mounted directly opposite each other near the center of the leg. In order to average the strain in each leg the gauges were wired in series.

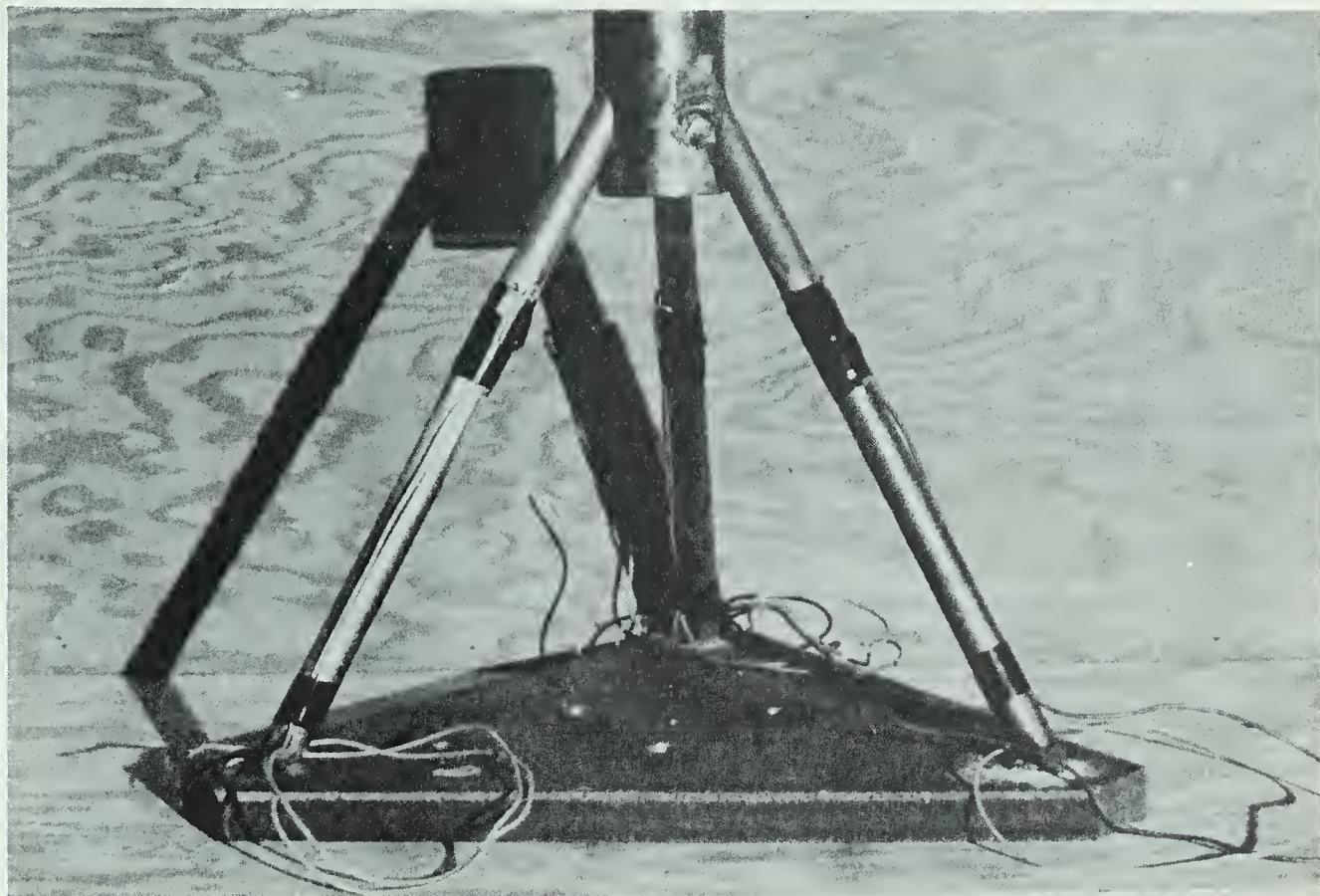


FIGURE 3.3 TYPICAL REACTION TRIPOD

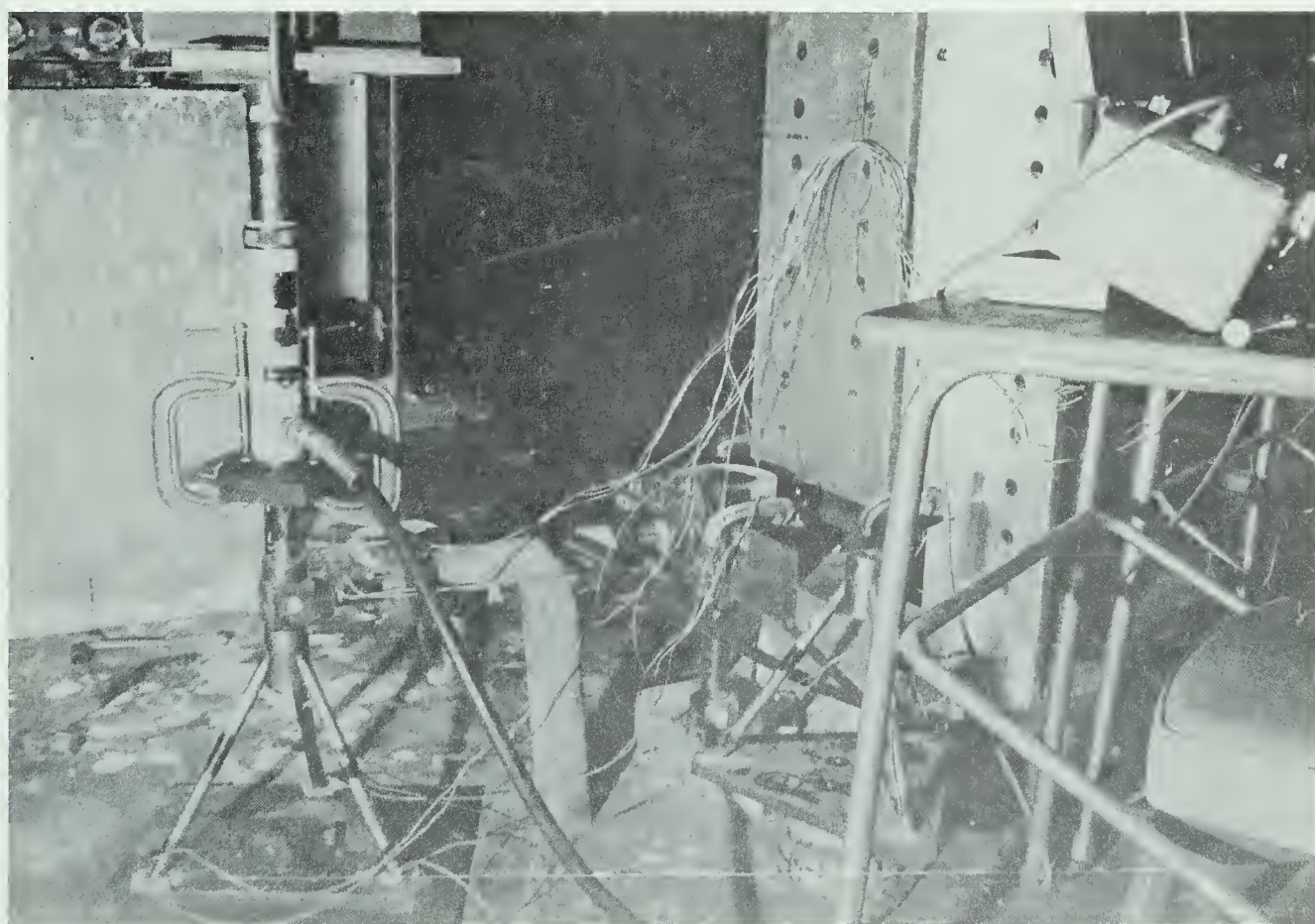


FIGURE 3.4 TRIPOD CALIBRATION SET-UP

The calibration was carried out in three known directions (vertical plus two perpendicular horizontal directions). The calibration set up is shown in FIGURE 3.4. The average strain in a given direction for a unit load was the preliminary dynamometer constant. By designating the three directions as 1, 2, 3 and each leg as 1, 2, 3, a set of 9 constants, 3 for each leg, is obtained as follows when load is applied in directions 1, 2, 3.

Direction Leg	1	2	3
1	c_{11}	c_{12}	c_{13}
2	c_{21}	c_{22}	c_{23}
3	c_{31}	c_{32}	c_{33}

When a force of unknown magnitude and direction was then applied on the dynamometer, three readings were registered such as D_1 , D_2 , D_3 . The magnitude and direction of this force could be determined by writing three equations for the three readings in terms of components of the force, in the directions of calibration 1, 2, 3.

$$D_1 = c_{11}P_1 + c_{12}P_2 + c_{13}P_3$$

$$D_2 = c_{21}P_1 + c_{22}P_2 + c_{23}P_3$$

$$D_3 = c_{31}P_1 + c_{32}P_2 + c_{33}P_3$$

In order to give the reactions directly, this matrix was inverted to give values of P_1 , P_2 , P_3 in terms of D_1 , D_2 , D_3 .

After calibration the tripods were placed into position on the top of concrete pillars as shown in FIGURE 3.1. The pillars were 18 inches square and 4-1/2 feet high. They were reinforced by four number four bars vertically and number three ties at the top and bottom. Two sections of 5" x 5" x 3/8" angle iron were cast into the bases of the

pillars in order to anchor them to the testing floor. Three 3/8 inch diameter threaded steel rods were cast into the top of the pillar to position the tripod.

The tripods were leveled and adjusted to the same elevation by means of non-shrink cement grout and a surveyor's level.

The slab was instrumented with 103 A-12 SR-4 electric strain gauges. A total of 54 were used on top steel and 49 on bottom steel. These were located with the hopes of determining distribution and magnitude of steel strains across lines of maximum moments in the panels. In order that the number was not too excessive, only one-quarter of the slab was fully instrumented. The locations of these are shown in FIGURES 3.5 and 3.6. These gauges were given three coats of water-proofing plus a coating of Epoxylite for protection.

Each of the strain gauges was connected to its own dummy gauge and the leads connected to a bank of rheostats. By means of these rheostats, a voltage could be impressed on the bridge circuit in order that the unbalance voltage was zero. These balancing units were then connected to a Dymec digital recorder. This unit automatically read the voltage changes in each of the bridge circuits and recorded the result on seven track magnetic tape and simultaneously printed the readings on a digital printer connected to the Dymec. The intention was to use the magnetic tape as a data source on the computer but problems arose necessitating the punching of the data on cards. For this reason not all tests have been analyzed. The Dymec recorder is shown in FIGURE 3.7.

Deflections were measured by twenty-three 0.001 inch dial gauges and ten electrical transducers. These were positioned as in

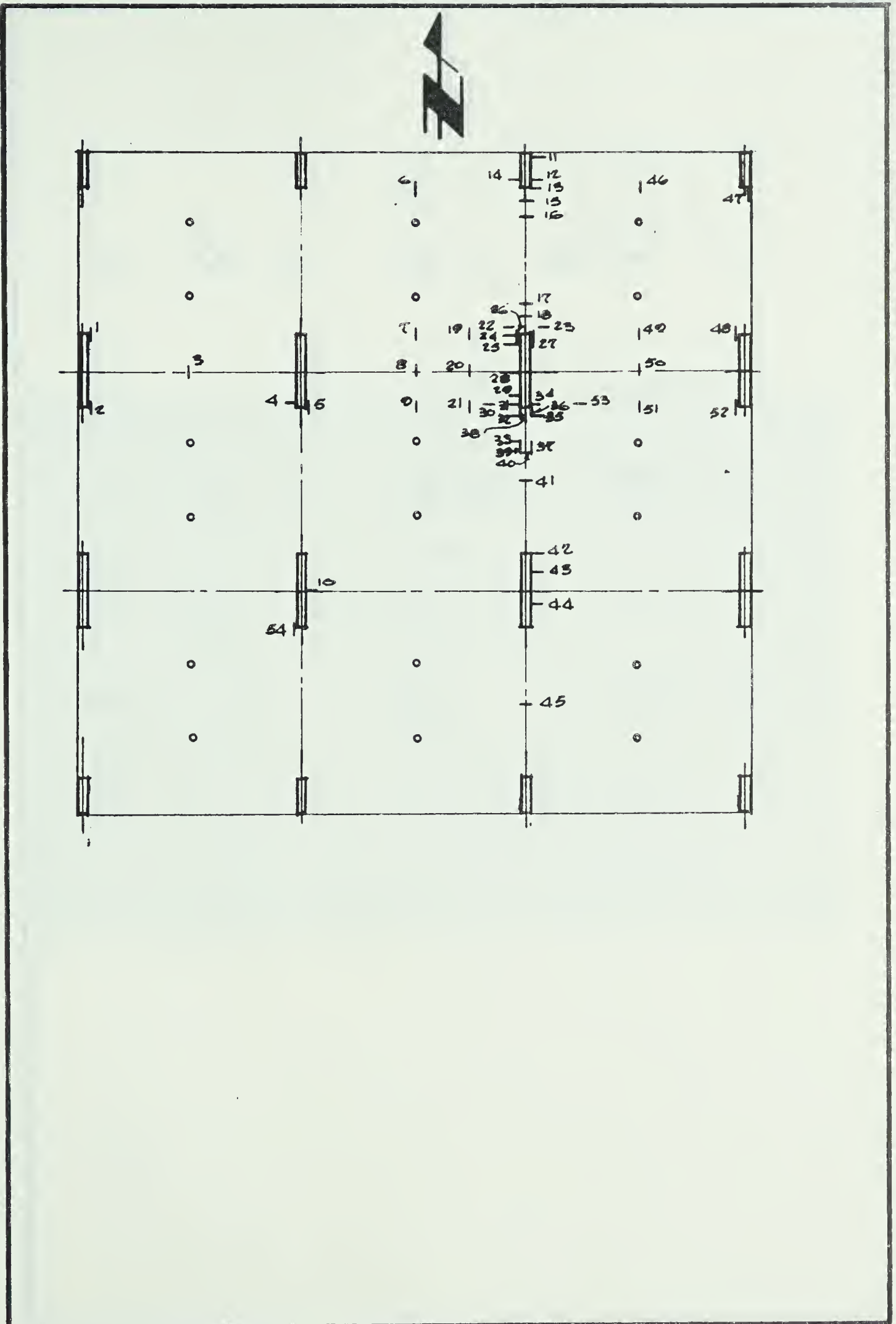


FIGURE 3.5 TOP STEEL STRAIN GAUGE LOCATIONS

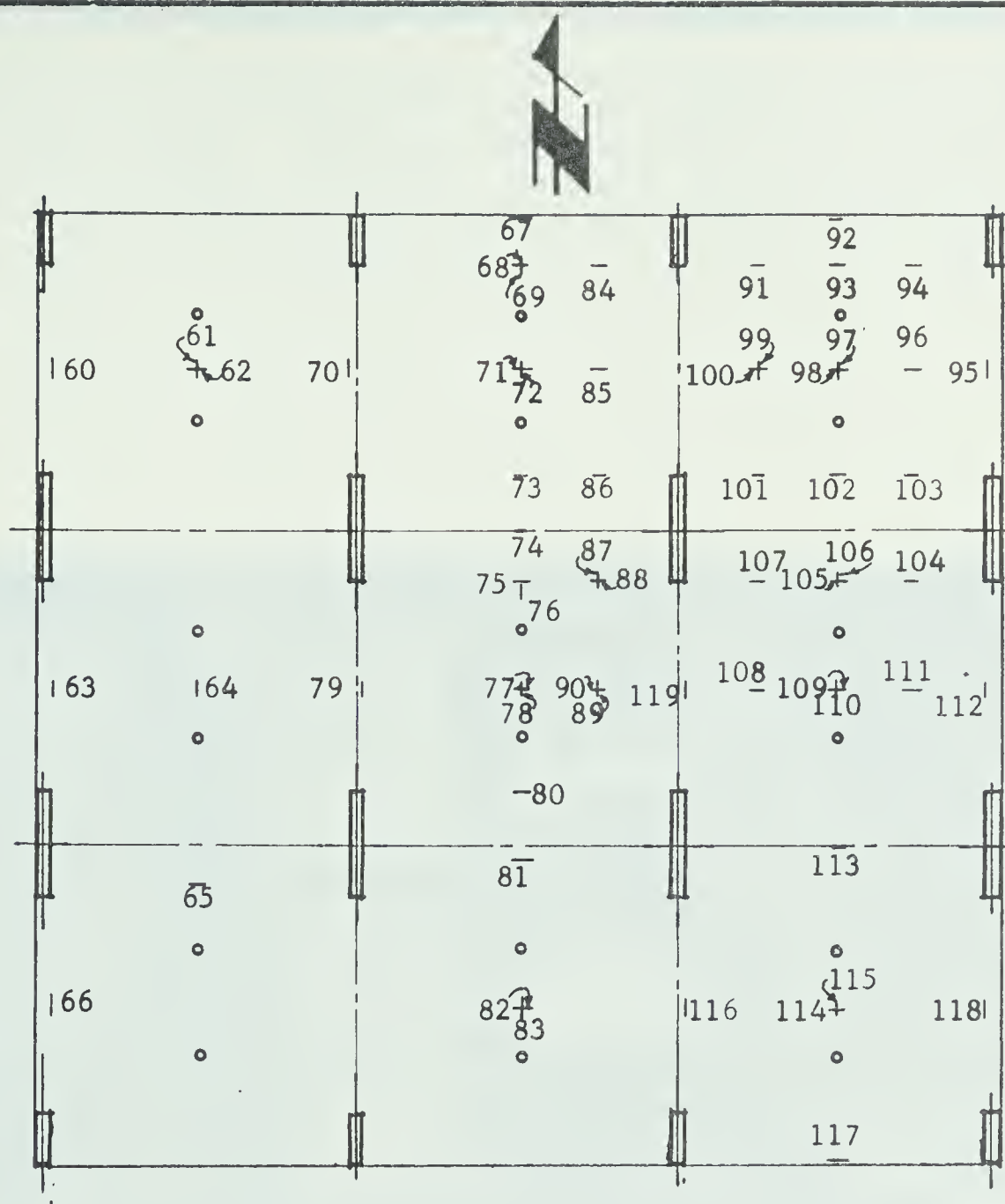


FIGURE 3.6 BOTTOM STEEL STRAIN GAUGE LOCATIONS

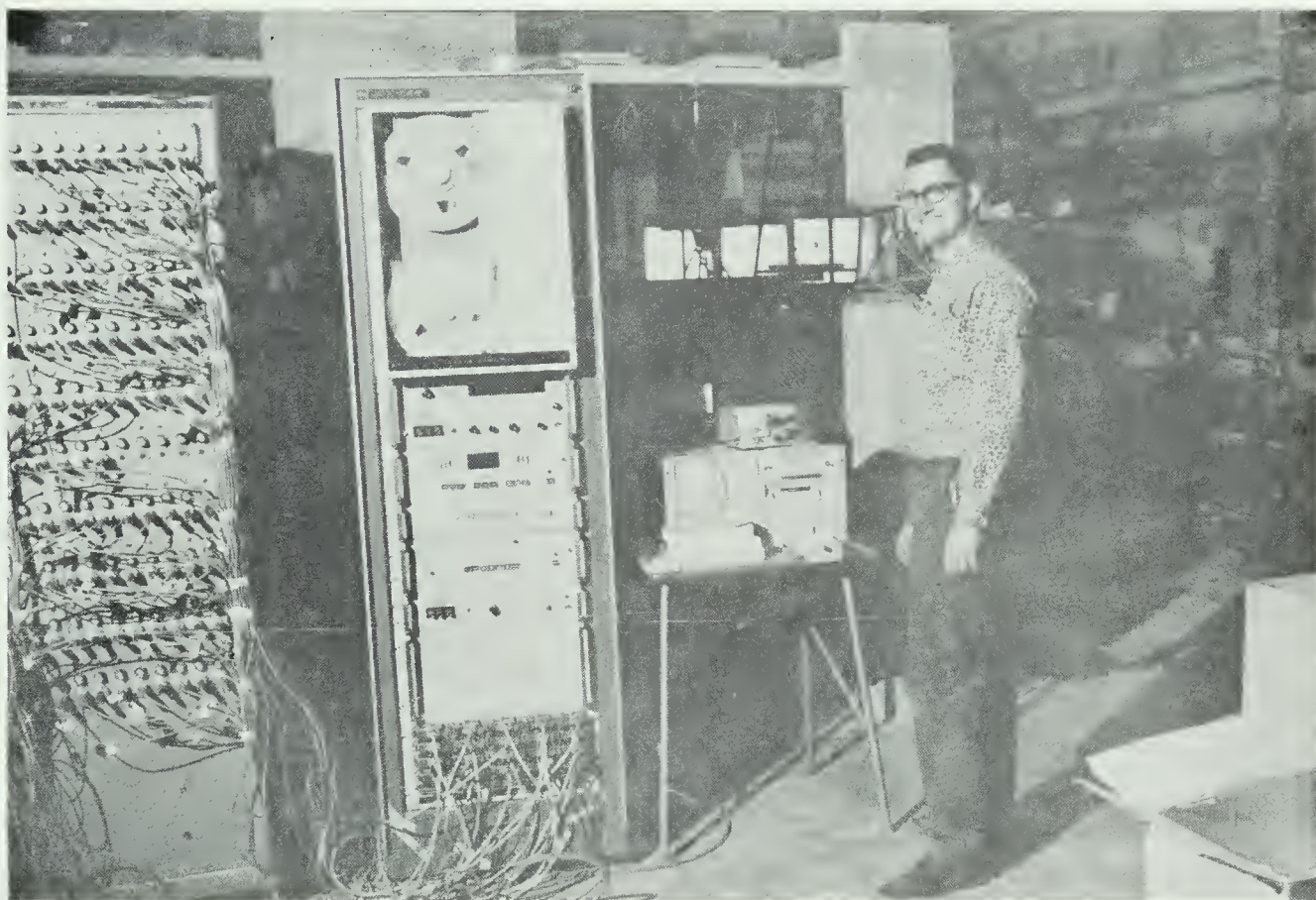


FIGURE 3.7 . DYMEC DATA ACQUISITION SYSTEM

FIGURE 3.8. The output signal from the transducers was also fed into the Dymec recorder. All deflection instruments were supported by magnetic base stands resting on 3" x 1-1/2" channels spanning between pillars.

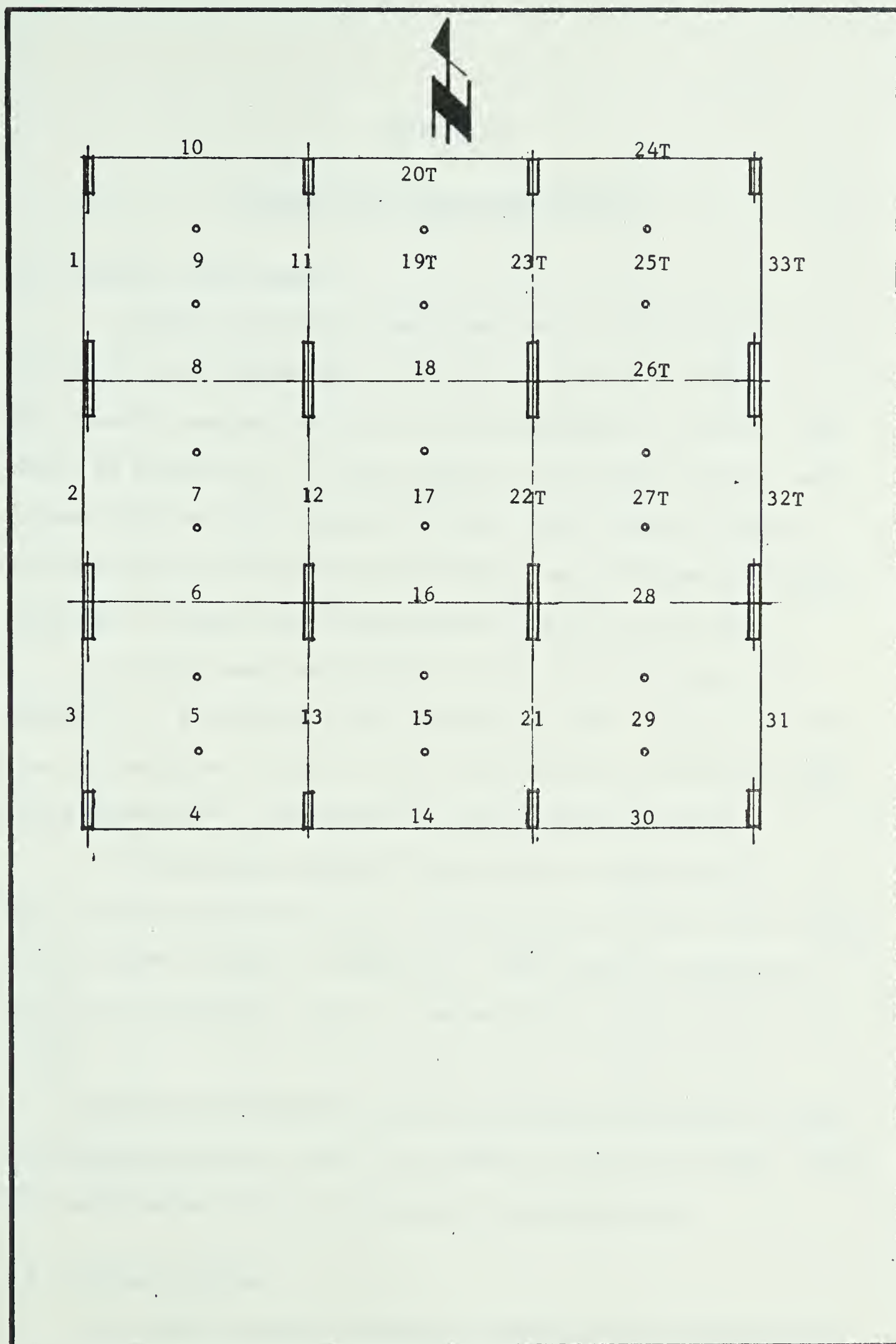


FIGURE 3.8 DIAL GAUGE AND TRANSDUCER LOCATIONS

CHAPTER IV

MOMENT-STEEL STRAIN RELATIONSHIPS

4.1 Design of Test Beams.

In order to derive a moment-steel strain relationship, two test beams were designed and constructed. In order to achieve the best possible results from only two test specimens, it was felt they should be reinforced in a similar manner to the model test slab and be made with the same concrete, 2" thick. Thus, with two types of reinforcing bars in the slab, the two beams were designed according to the most prevalent use of each type of bar in the test slab.

The test beam incorporating the 1/8 inch square bars corresponds to a bar spacing of 1/8 squares @ 2-3/8". The width of the beam was sufficient to give a total of six bars at this spacing plus the required cover. This beam had a total width of 12-3/8".

The second beam used 1/4" round bars at a spacing of 3". Again six bars were used to give a total width of 15-1/2". The layout of these beams is shown in FIGURE 4.1. Both beams had temperature reinforcing equivalent to that in the model slab, 1/8 inch squares at 3-7/8".

Each main reinforcing bar was instrumented with one A-12 SR-4 strain gauge which was placed at the center of the bar's length. These were waterproofed similar to the bars in the model slab.

4.2 Testing of Beams.

The moment-curvature beams were tested the day following the

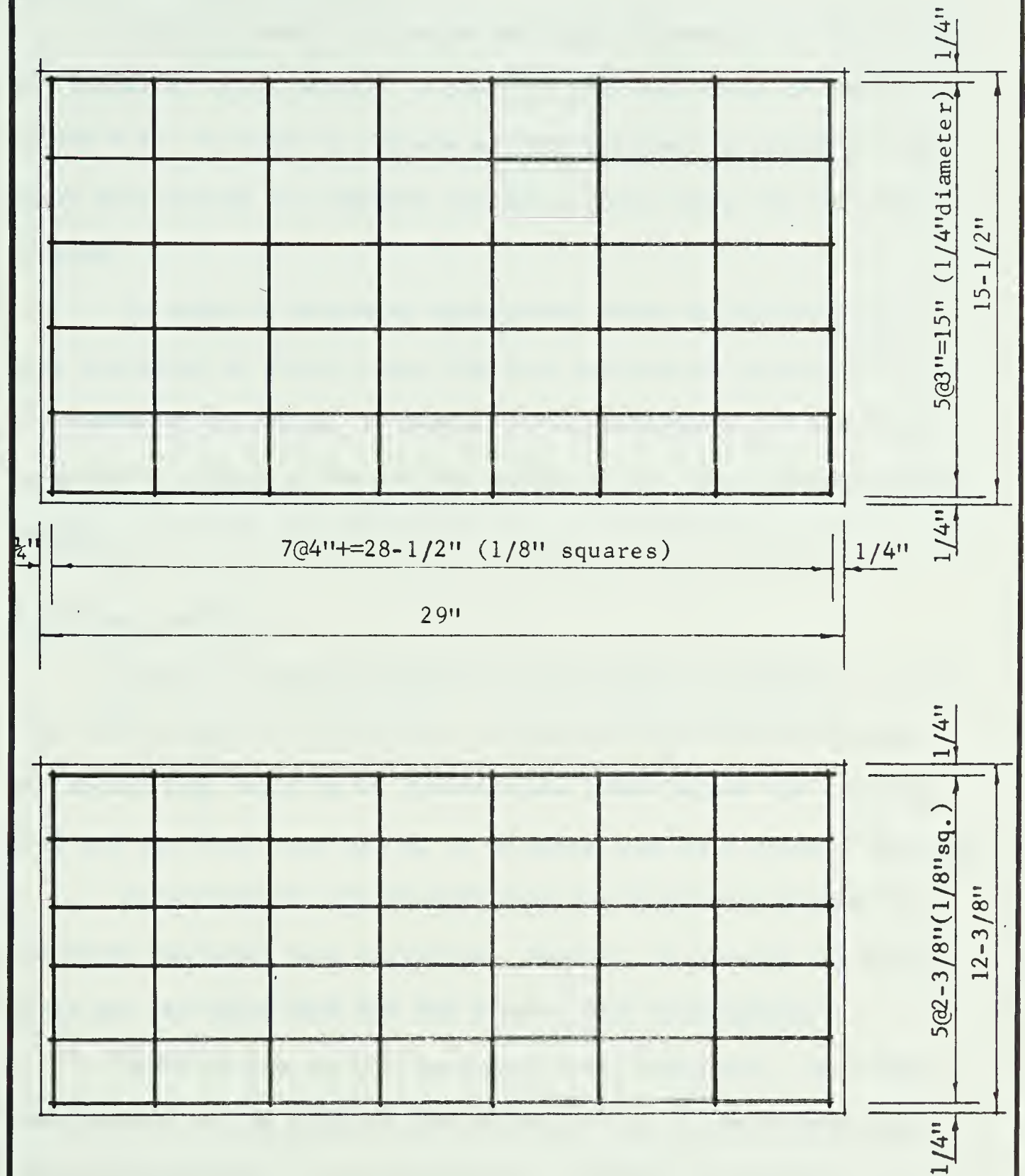


FIGURE 4.1 LAYOUT OF MOMENT-STEEL STRAIN BEAMS

final test on the model slab. The properties of the concrete at this stage are seen in TABLE 4.1.

The test beams were supported on a 24" center to center span and loaded at third points. A photo of the test setup is shown in FIGURE 4.2. In order to achieve as much accuracy as possible, the beams were tested in a Baldwin testing machine using the low range of load.

In order to determine curvatures, three deflection gauges were installed at points under the load application points and at the center of the span. By geometric considerations, it was then possible to calculate the average curvature over the constant moment region.

4.3 Test Results.

Plans for testing involved three stages of loading. The initial test was at low load, up to working load but with no cracking. The second test would be at intermediate loads beyond the cracking load and the final test was up to ultimate load on a cracked specimen.

Unfortunately, the 12-3/8" beam was loaded too rapidly and the first two tests were destroyed. However, an attempt was made to carry out the third test but the results were meaningless.

Tests on the 15-1/2" beam were more successful. The tests were carried out as planned, and it was not until the results were being reduced that it was noticed that cracking was present in the first test. Results of this test are shown in FIGURE 4.4 in the form of a moment-steel strain relationship.

TABLE 4.1
CONCRETE STRENGTHS

Cylinder Number	Age at Test (Days)	Compressive Strength (psi)	Splitting Strength (psi)	Modulus of Elasticity (psi)
1	28		341	2.54×10^6
2	28	3610		
3	28		257	2.70×10^6
4	28	3540		
5	43	3700		2.74×10^6
6	43	3650		
7	43		349	
8	43		310	
9	28		252	3.30×10^6
10	28	3400		
11	28		339	
12	28	3470		
13	8	2550		
14	8	2690		
15	8	2740		
16	8	2630		
17	50	3740		
18	50	3780		
19	50	3860		
20	50	3860		

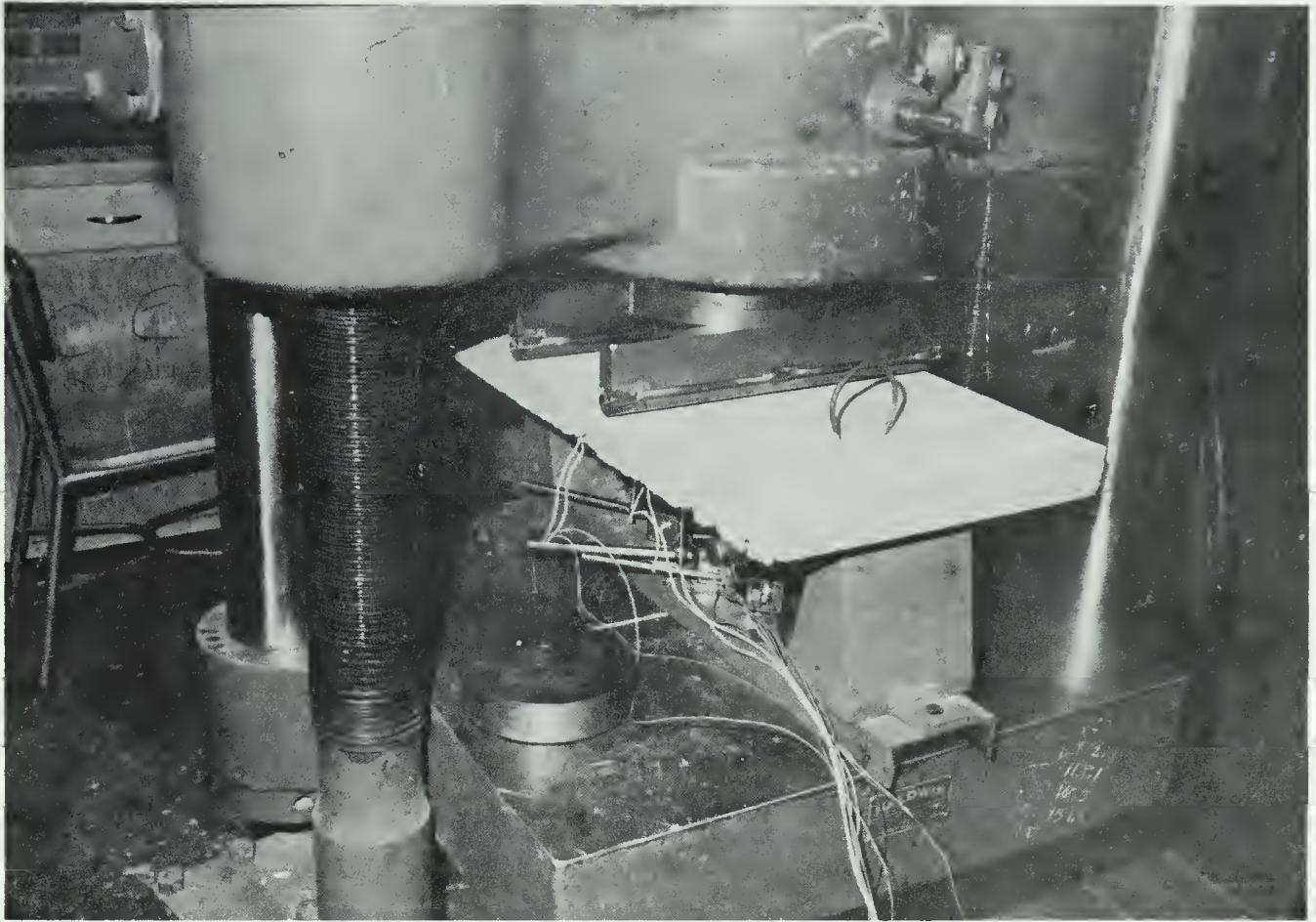


FIGURE 4.2 MOMENT-STEEL STRAIN TEST SET-UP

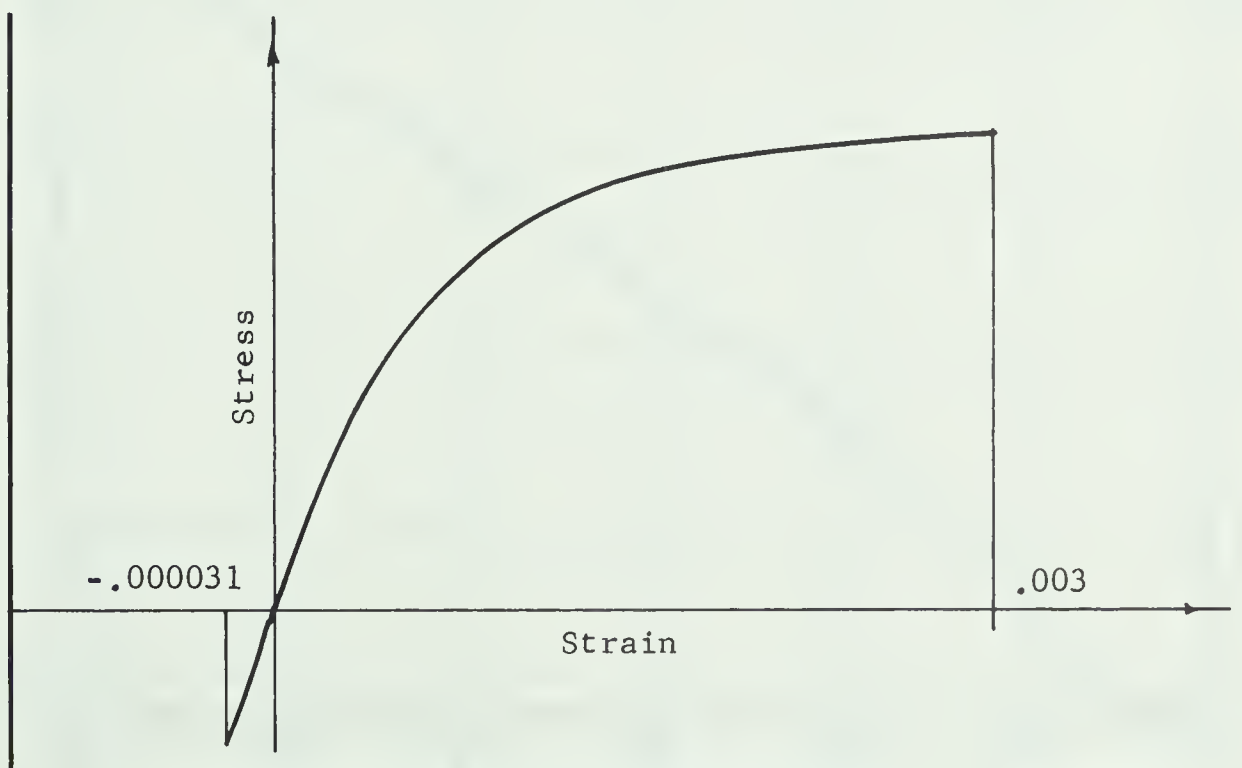


FIGURE 4.3 STRESS-STRAIN CURVE FOR CONCRETE AS USED IN THEORETICAL MOMENT-STEEL STRAIN RELATIONSHIPS

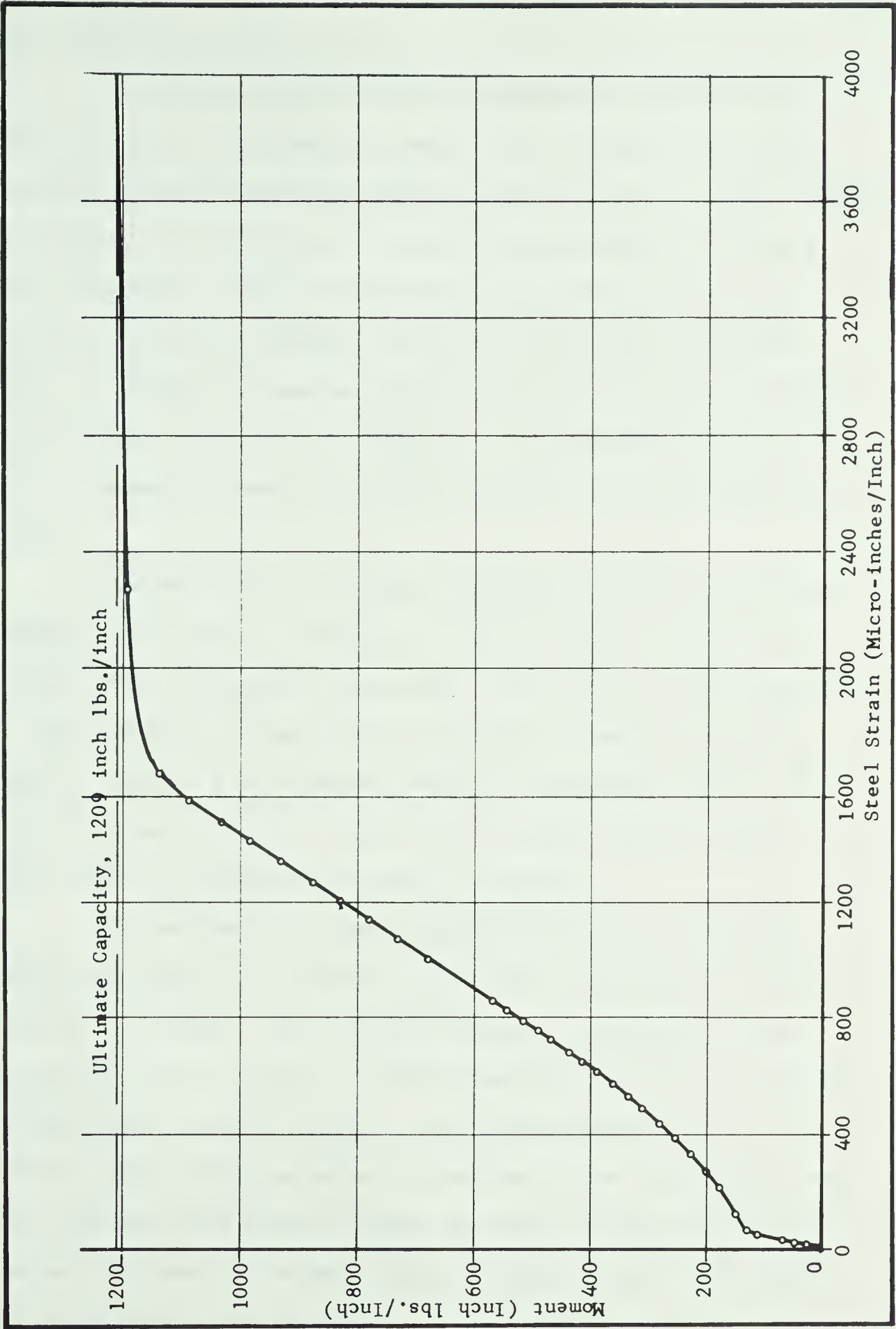


FIGURE 4.4 EXPERIMENTAL MOMENT-STEEL STRAIN RELATIONSHIP FOR 1/4" ROUND BARS IN 2" SLAB

4.4 Reduction of Data.

By superimposing the test to failure on the cracked beam on the first test to intermediate loads, a plot of moment-steel strain was derived which should approximate closely the case of virgin loading to failure. By idealizing this curve it was possible to develop a set of relations useful for calculating the moment from any steel strain in a 1/4 inch diameter bar for the first test on the model slab. Similarly, by superimposing the results of the test to failure on the second test on the test beams, it was possible to develop a plot of moment vs. steel strain for the case of a fully cracked test beam.

The relationship for virgin loading contains an interesting segment which should be explained. The plot starts as would be expected, with a relatively steep slope. Upon initial cracking there is a sharp increase in steel strain for a given increase in applied moment. Then at a point where cracking is presumably complete, the relative steel strain for a given applied moment decreases again. This slope is continued until yield is reached.

The explanation of this "cracking transition" zone can be easily explained if the behavior of stresses is examined on a cross-section in the beam. Upon initial cracking, a large jump in steel stress must occur in order to continue equilibrium due to the loss in tensile force from the concrete. As cracking progresses, the steel takes an ever increasing amount of the load, but at a decreasing rate. The cracking of the concrete drives the neutral axis up and thus increases the lever arm between the steel tension force and the center of compression in the concrete.

When the progression of the cracks has reached the neutral axis, the lever arm changes no more and the relationship between steel strain and applied moment again takes an upward step in slope until yield strains are encountered in the steel.

4.5 Theoretical Analysis.

Because there were no results obtained for the 12-3/8" test beam, no relationship could be produced for moments from steel strains. This posed a problem in the analysis for bending moments in the slab since the majority of strain readings obtained during testing were derived from the 1/8 inch square bars.

This prompted interest in a theoretical analysis to derive the relationship. Fortunately, a computer program of this type was available in the department which required only minor changes in order to produce the desired results. Basically, this program is for the calculation of moment-curvature relationships in axially loaded members. However, it was possible to set the axial load equal to zero and consider only uniform moment.

The calculations in this program are based on a concrete stress strain curve as shown in FIGURE 4.3. The compression section of the curve follows the relationship

$$f_c = 2f_c'' \left(\frac{\epsilon}{\epsilon_o} \right) - 1.03 f_c'' \left(\frac{\epsilon}{\epsilon_o} \right)^{1.67}$$

where

$$\epsilon_o = 2f_c''/E_c$$

$$E_c = \text{Experimental Value}$$

$$f_c'' = 0.85 f_c'$$

$$\epsilon_u = 2 \epsilon_o = 0.004$$

The tension section of the curve follows the relationship

$$f_t = 2f_t'' \left(\frac{\epsilon}{\epsilon_{ut}} \right) - \left(\frac{\epsilon}{\epsilon_{ut}} \right)^3$$

where

$$\epsilon_{ut} = 2f_t''/E_c = -0.000031 \text{ in./in.}$$

$$f_t'' = -.325 f_c'$$

The stress-strain curve for steel reinforcing was assumed elastic - perfectly plastic.

The program considers the cases of pure axial load (which is zero in this case), compression failure, tension failure with no cracking and tension failure on a cracked section.

The results of this analysis are shown in FIGURES 4.5 and 4.6 in the form of applied moment versus steel strain plots. These were idealized and used later to analyze the strain readings measured during testing.

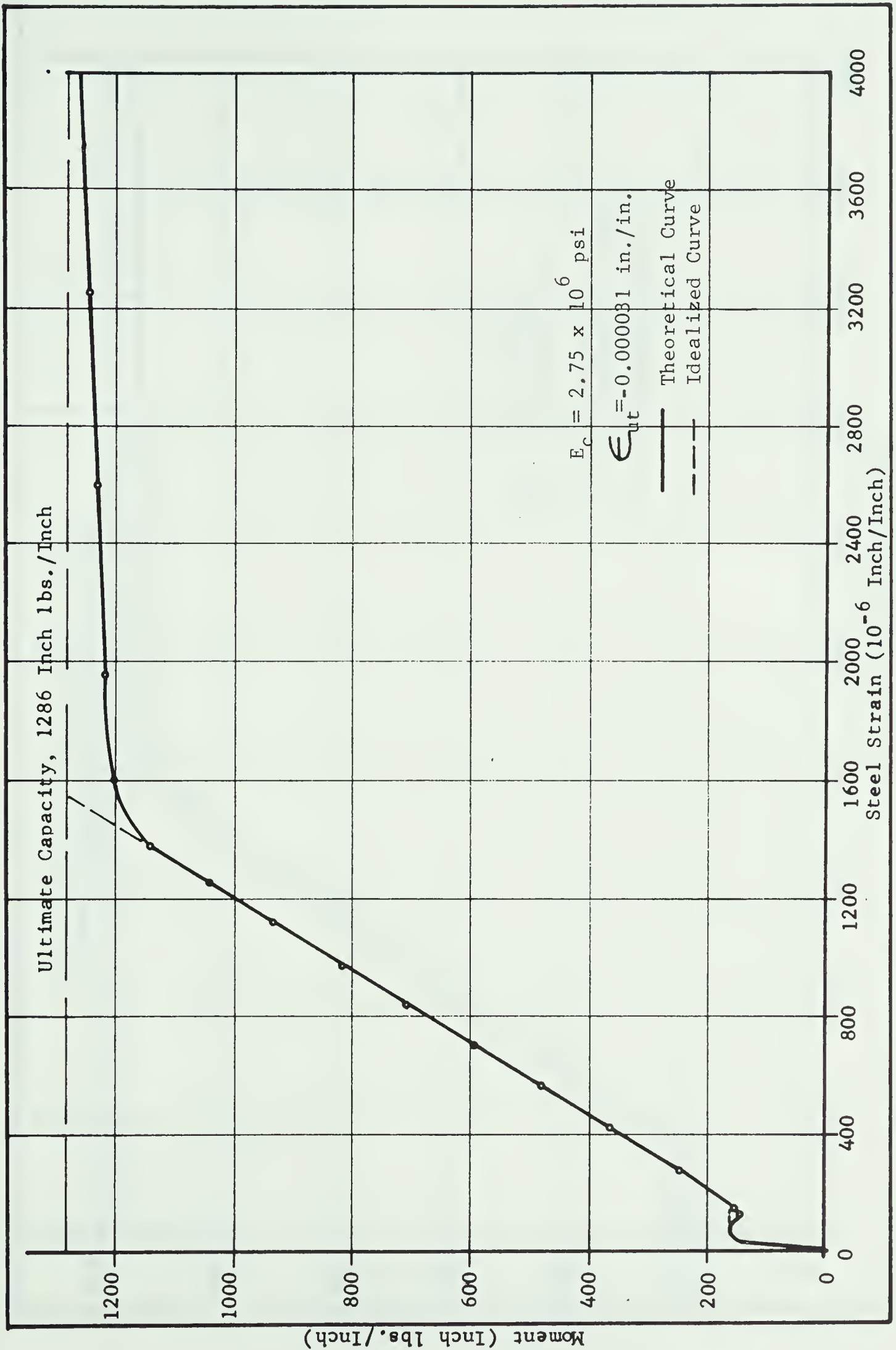


FIGURE 4.5 THEORETICAL MOMENT-STEEL STRAIN RELATIONSHIP FOR 1/4" BARS IN 2" SLAB

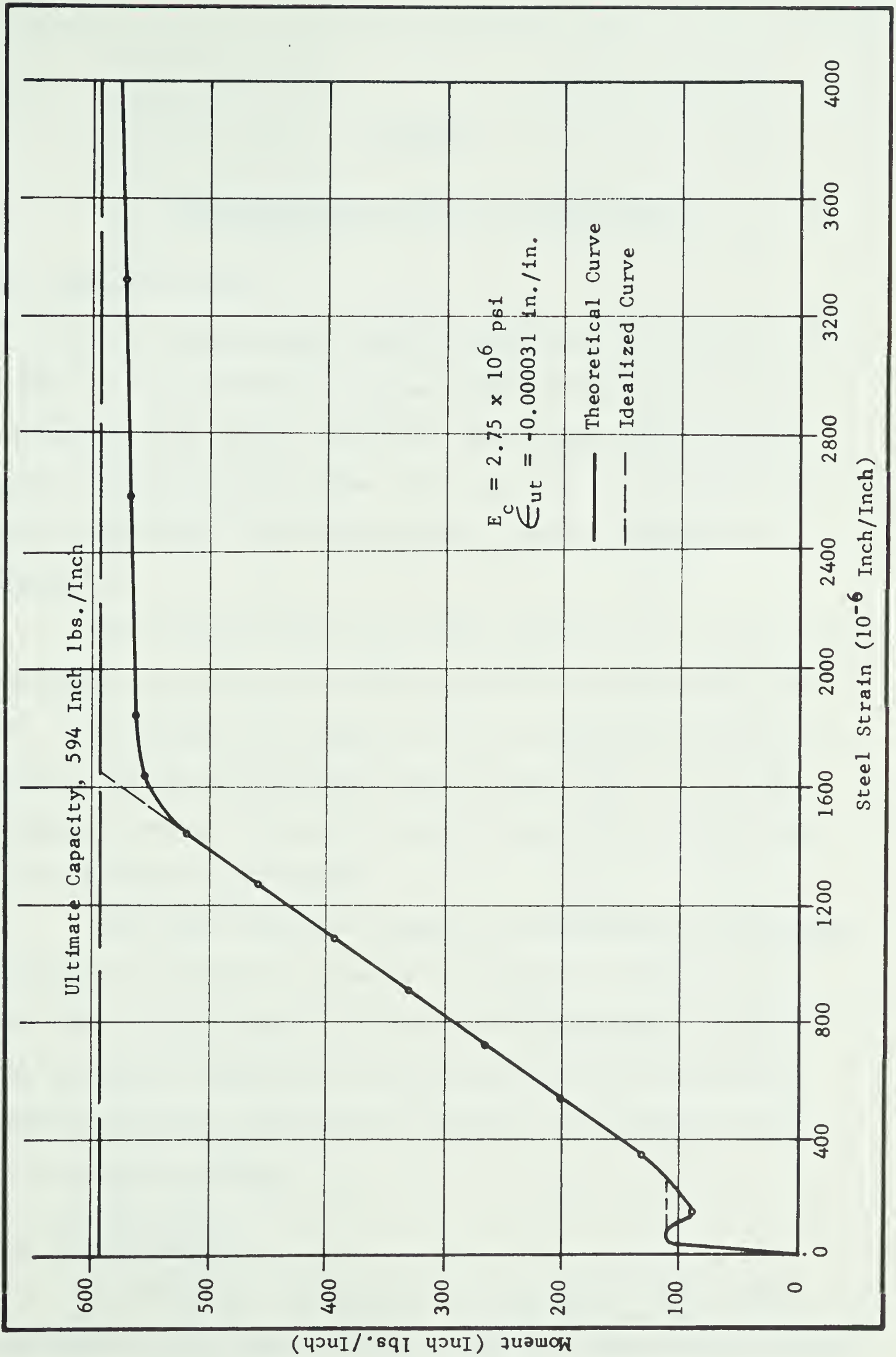


FIGURE 4.6 THEORETICAL MOMENT-STEEL STRAIN RELATIONSHIP FOR 1/8 INCH SQUARE BARS

CHAPTER V

PRESENTATION AND ANALYSIS OF TEST RESULTS

5.1 Outline of Tests.

The testing program involved eight tests, differing from one another in the magnitudes of applied loads. Three of the tests were carried out under uniform load on all panels while four considered various strips of panels loaded. The last test was a loading of the center panel only. The loading patterns used are presented in FIGURE 5.1.

Since the dead load of the model structure did not scale from the prototype structure, the total applied load during testing was not all "live load." The weight of the load distribution system only added 15 psf to the 25 psf slab dead load which left 35 psf to be applied to the slab through the loading system before the dead load of the structure was attained.

This factor should be remembered when examining the magnitudes of deflections which are presented on the basis of applied load to the model structure rather than as live load deflections. However, the increments of deflections would be realistic for corresponding loading increments, providing the load does not reach such a level as to initiate cracking.

5.2 Test Procedure.

In all cases, an initial set of readings were taken before application of the load. In Tests 1 to 6, the load was applied incre-

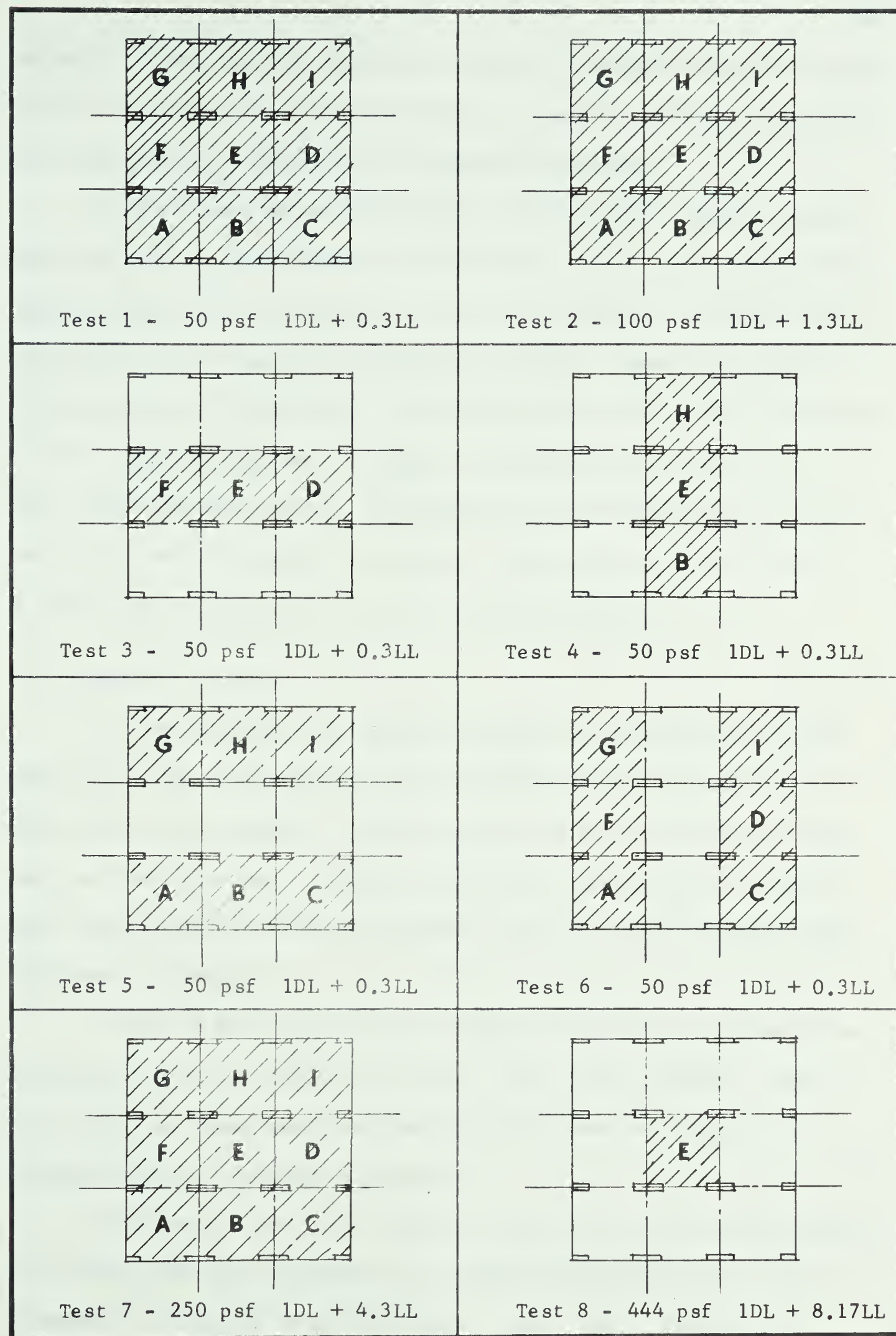


FIGURE 5.1 DIAGRAM OF LOADING PATTERNS

mentally to some desired level with a set of readings obtained at each level of load, and then removed using the same increments. After the load was removed, another set of readings were taken.

During testing the slab behavior and most of the deflections were recorded manually while the slab steel strain, tripod leg strain, applied load and the remainder of the deflections were recorded automatically on the Dymec Data Acquisition System. Immediately after the application of each load increment the time and hydraulic pressure on the loading system were recorded. Deflections for Tests 3 to 7 were read immediately after an increment of load application and just before the next increment was applied. The deflections for Tests 1, 2 and 8 were recorded only once for each load level.

5.3 Reduction of Data.

During testing, all results obtained automatically from the Dymec Data Acquisition System were recorded on magnetic tape for later use in the computer. Unfortunately the tape could not be read and was thus of no use. Luckily the results were printed on paper tape simultaneously with the recording, and from these, punched cards were easily produced.

Since it was hoped that the magnetic tape would eventually be available, only the results from Tests 2 and 7 were punched. Thus, only these two tests were analyzed using the computer program for determination of reactions and moments.

The results from this computer program were quite discouraging. All of the reactions appeared to be completely meaningless for all increments of loads during both tests. This, then, ruled out any

hopes of making a static check on summations of reactions and calculations of total moments along lines in the slab based on the reactions.

Generally, the bending moments computed from the strains in the reinforcing bars in the slab were erratic or meaningless. This was due to the erratic strains recorded by the Dymec. The problem of inconsistency linked to the problem of a certain percent of the strain gauges being faulty in operation led to the impossibility of predicting bending moment distributions along lines in the slab as intended. Therefore the emphasis is one of general behavior with considerations being given to deflections and ultimate capacity of the slab.

5.4 Test 1.

Test 1 consisted of a uniform load on all panels up to a maximum of 50 psf ($1DL + 0.3LL$) in increments of 10 psf and subsequent incremental removal of loading. This loading corresponds to a live load of 15 psf on the prototype structure due to the scaling of the dead load.

Basically, this test was to give an indication of the working condition of the equipment without causing inelastic deformations or cracking in the structure. Both deflections and strains were recorded. Deflection readings for initial increments were non-linear indicating that the slab settled unevenly due to seating of the support mechanism. The strain readings obtained from the Dymec were erratic. Steps were taken to correct this fault and subsequent tests were performed believing the equipment was functioning properly but analysis of later test results showed that it was a fault of the recording unit rather than in the test structure instrumentation.

Due to the erratic nature of the readings, and that the primary purpose of this test was to examine the behavior of the instrumentation, the results of this test are not presented. No cracks were observed after the test and it was thus assumed that the slab was uncracked for subsequent tests.

5.5 Test 2.

Test 2 was similar to Test 1 except that the applied load was taken to 100 psf (1DL + 1.3LL).

For each deflection gauge, a plot of applied load versus deflection was prepared. A representative selection of these is presented in FIGURE 5.2. The location of the deflection gauges is shown in FIGURE 3.8.

Initial examination of these plots would indicate cracking of the structure at loads of as low as 60 psf applied load (0.5LL) and large residual deflections. These results are probably misleading since the break off in the curves is due to creep of the structure rather than cracking. Thus the slope is probably still a straight line until higher loads plus the action of creep at each load level. For this reason, subsequent tests involved the recording of deflections at the beginning and end of each load level along with the time of observations.

First cracking in the structure was observed on the top surface of the slab between columns along lines parallel to the long faces of the columns, at an applied load of 80 psf (0.90LL). The final crack pattern is shown in FIGURES 5.3 and 5.4.

This observation is supported by the increased rate of deflec-

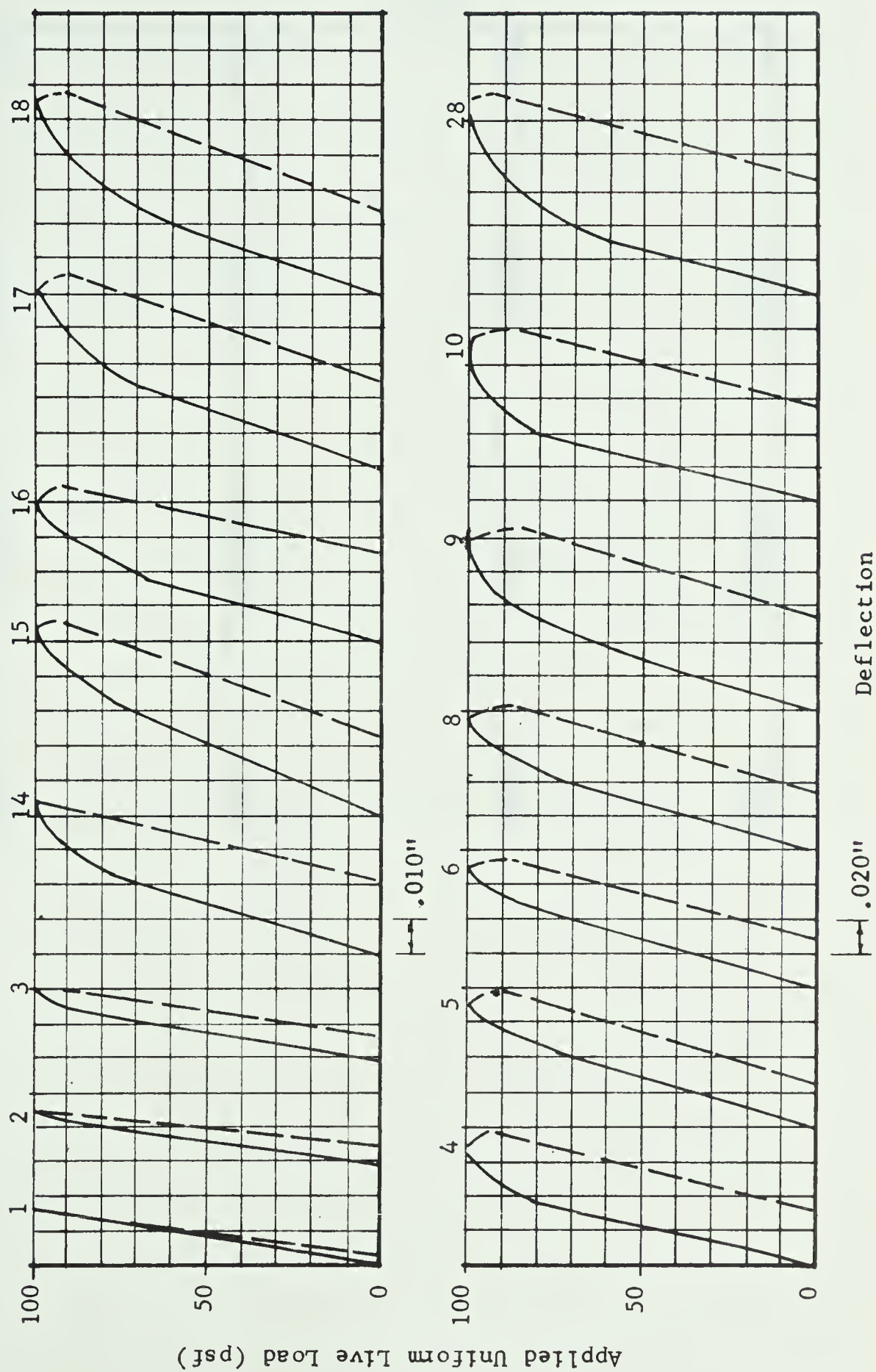


FIGURE 5.2 LOAD-DEFLECTION CURVES FOR TEST 2

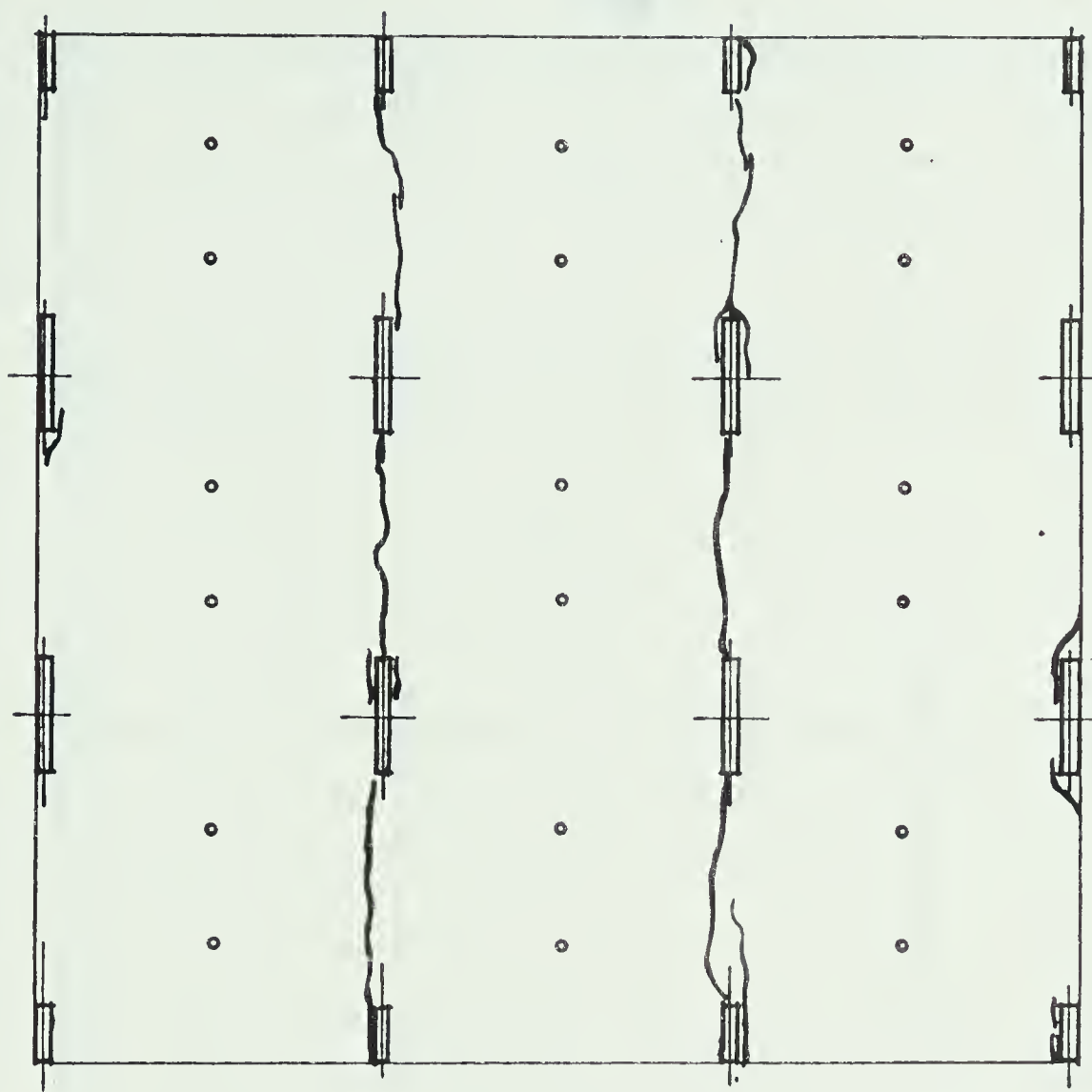


FIGURE 5.3 TOP CRACKS AFTER TEST 2

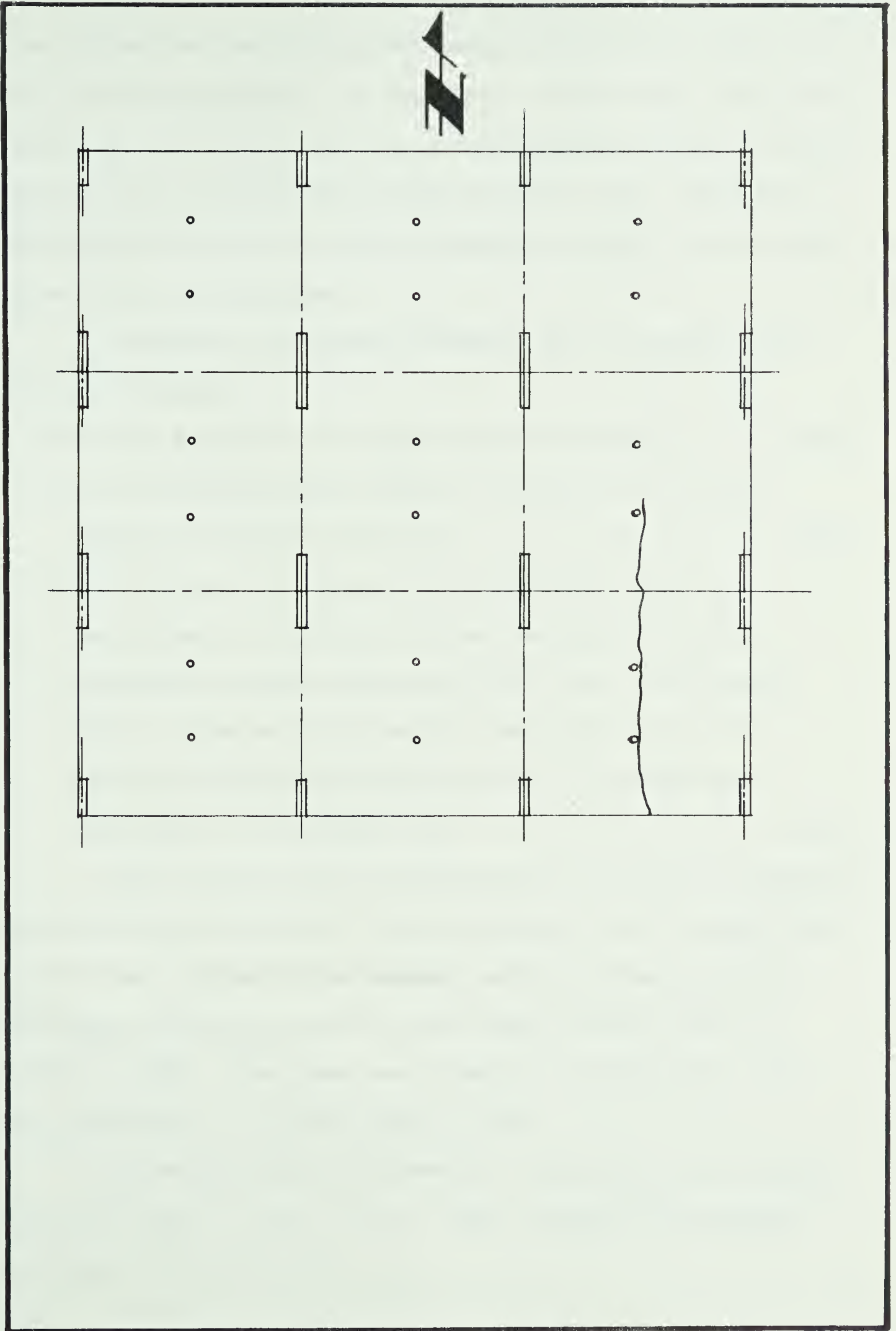


FIGURE 5.4 BOTTOM CRACKS AFTER TEST 2

tion with applied load for the dial gauges lying along the panel center-lines which are parallel to the long faces of the columns. The deflections at points on the lines joining the columns and parallel to their long faces do not show this increased deflection rate. These facts would tend to indicate the slab was behaving more like a one-way slab system than a two-way system.

According to the National Building code, the maximum limits for deflections are:

- a) For roofs which do not support plastered ceilings L/180
- b) For roofs which support plastered ceilings or for floors
which do not support partitions L/360
- c) For a floor or roof construction intended to support or
be attached to partitions or other construction likely to
be damaged by large deflections of the floor, the allowable
limit for the sum of the immediate deflection due to live
load and the additional deflection due to shrinkage and
creep under all sustained loads L/360

Since the creep was not evaluated in Test 2, only the immediate deflections may be analyzed. Thus only limits a and b are applicable. For this test, the deflection exceeded neither of these allowables. The maximum deflection occurred at dial gauge location 9 and was 0.097", or L/741. Thus there was no danger of failure due to immediate deflections at a maximum load of 1.3LL.

The maximum deflection observed at the center of the interior panel was 0.046" or L/1563. This is about one-half of the maximum deflection in the structure.

As would be expected, points on the column center lines deflected less than the panel center points. However, the points on the center lines running parallel to the long faces of the columns deflected only a small fraction of that observed at points on center lines perpendicular to the long faces of the columns.

In all cases a residual deflection was observed which amounted to as much as 60% of the total deflection. A high value of residual seemed to coincide with locations of increased deflections due to nearby cracking of the slab. In cases where cracking was not evident, the residual only reached about 33%.

Judging by the observed behavior of the slab during the tests, this residual is due to creep in the concrete while maintaining the load at a particular value. Since creep is a function of applied stress, it would seem that the relative amounts of creep would be greatest at locations of high bending moments. This can be seen to hold true when referring to FIGURE 5.2. Points 8, 9 and 10 and 28 have the largest residuals, which correspond to maximum positive moments in the outer panels. Since the negative moments occur at points of support, these locations do not deflect, although the effect of creep at these points is projected to the midspan positions.

5.6 Strip Loading Tests.

A series of four strip loading tests were performed in an attempt to produce maximum bending moments at critical sections in the structure. Since the strain readings have no meaning, little can really be gained from these tests. However, some concept of the effects of loaded spans on unloaded neighbouring spans can be seen by the relative deflections of the spans.

Test 3 considered the loading of the three panels on the north-south center line of the structure (panels DEF) as shown in FIGURE 5.1, to a maximum load of 50 psf ($1DL + 0.3LL$).

A representative selection of load-deflection plots is presented in FIGURE 5.5. As can be expected, some gauges showed almost no deflection, some points moved upward and some deflected downward. Behavior in every case is linear throughout the test, indicating no new cracking of the structure, which agrees with observations.

Test 4 was a loading of the panels on the east-west center line of the structure (panels BEH) to a load of 50 psf. Again, no new cracking was observed. Typical load-deflection curves are presented in FIGURE 5.6.

Test 5 was a loading of the two outside panel strips in the north-south direction (panels ABC and GHI) to a load of 50 psf. Typical load-deflection plots are presented in FIGURE 5.7. No new cracks were observed.

Test 6 was also a loading of the outside strips of panels, except in the east-west direction. Again, no new cracks were observed. Typical load-deflection plots are presented in FIGURE 5.8.

The maximum deflections of the centers of the four types of panels found in the structure are listed in TABLE 5.1 for each of the strip loading tests plus Test 2. Also included is a table indicating the percentage of the deflection of a particular point for any loading pattern to the deflection of the same point for some other loading pattern.

This table indicates that the application of a strip load on the inner panels in the direction parallel to the long faces of

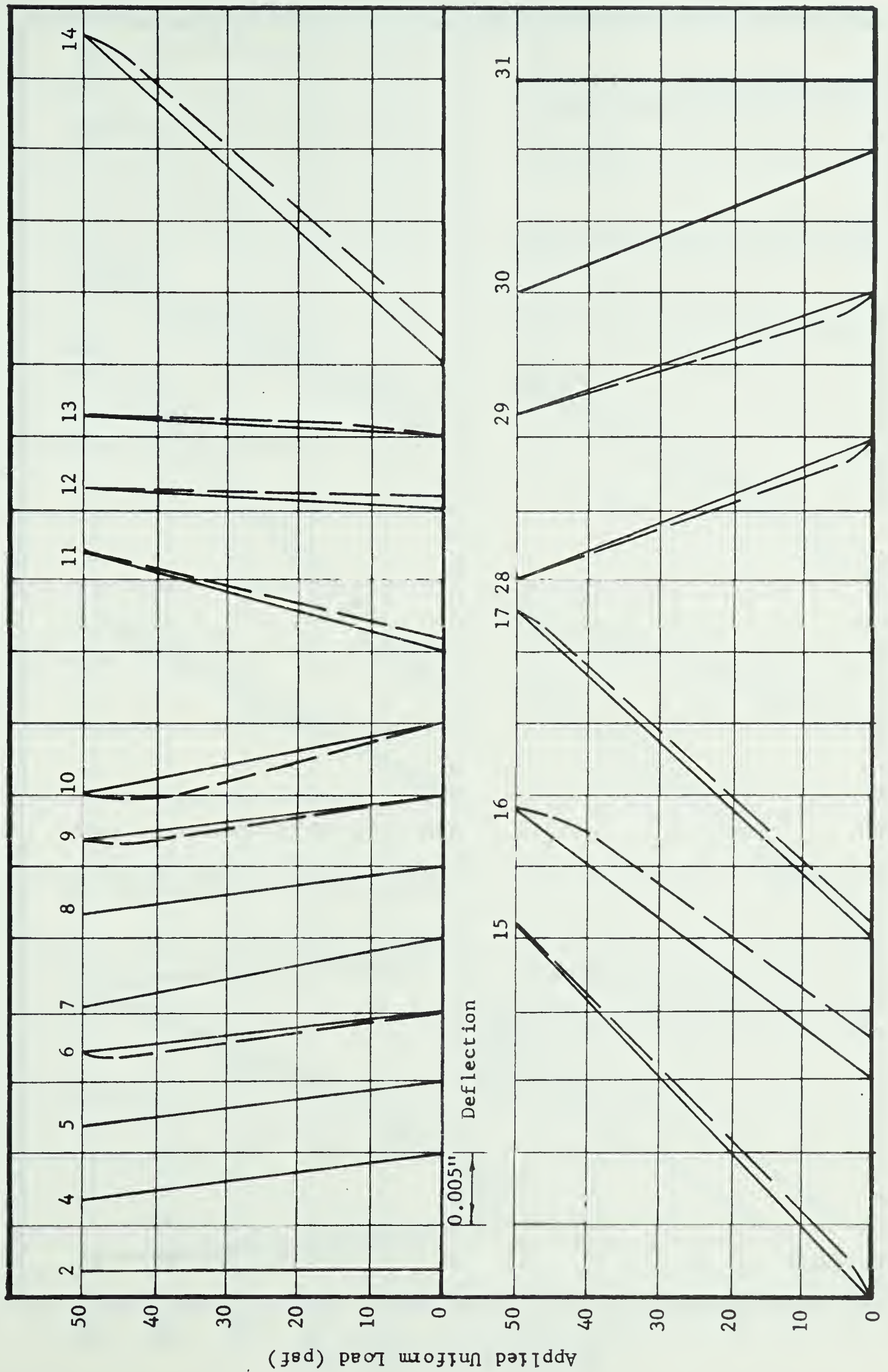


FIGURE 5.5 LOAD-DEFLECTION PLOTS FOR TEST 3

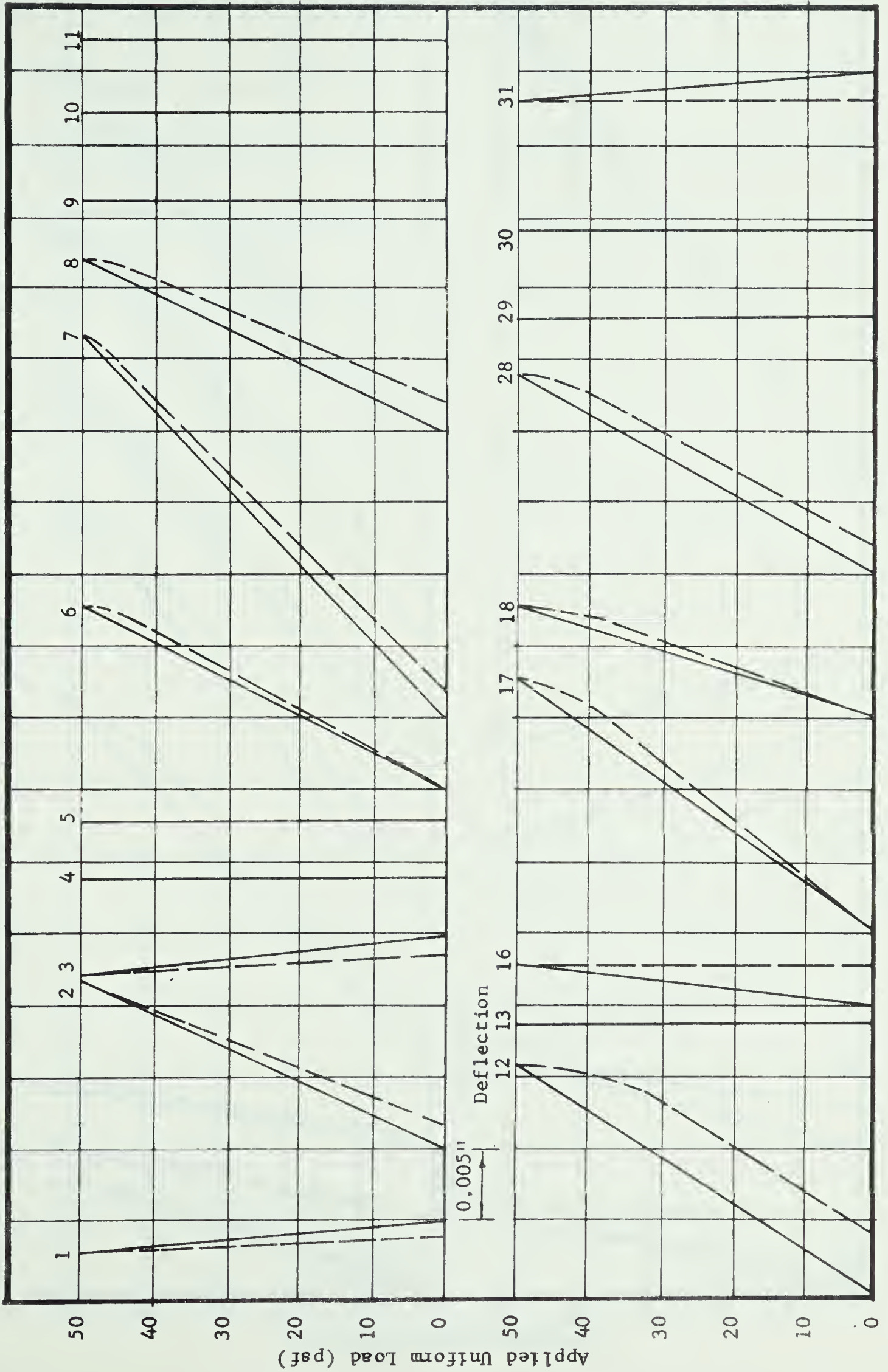


FIGURE 5.6 . LOAD-DEFLECTION PLOTS FOR TEST 4

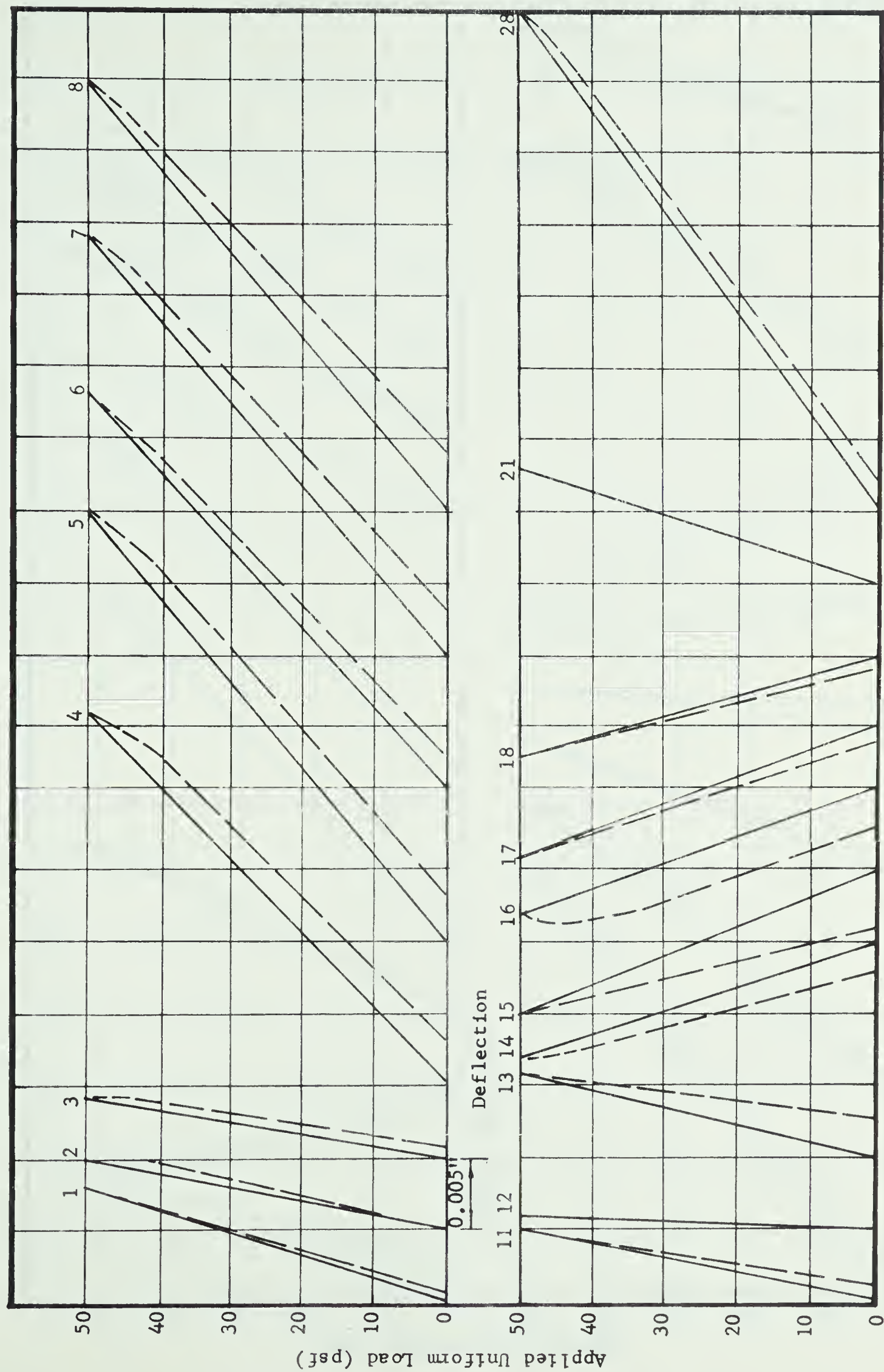


FIGURE 5.7 LOAD-DEFLECTION PLOTS FOR TEST 5

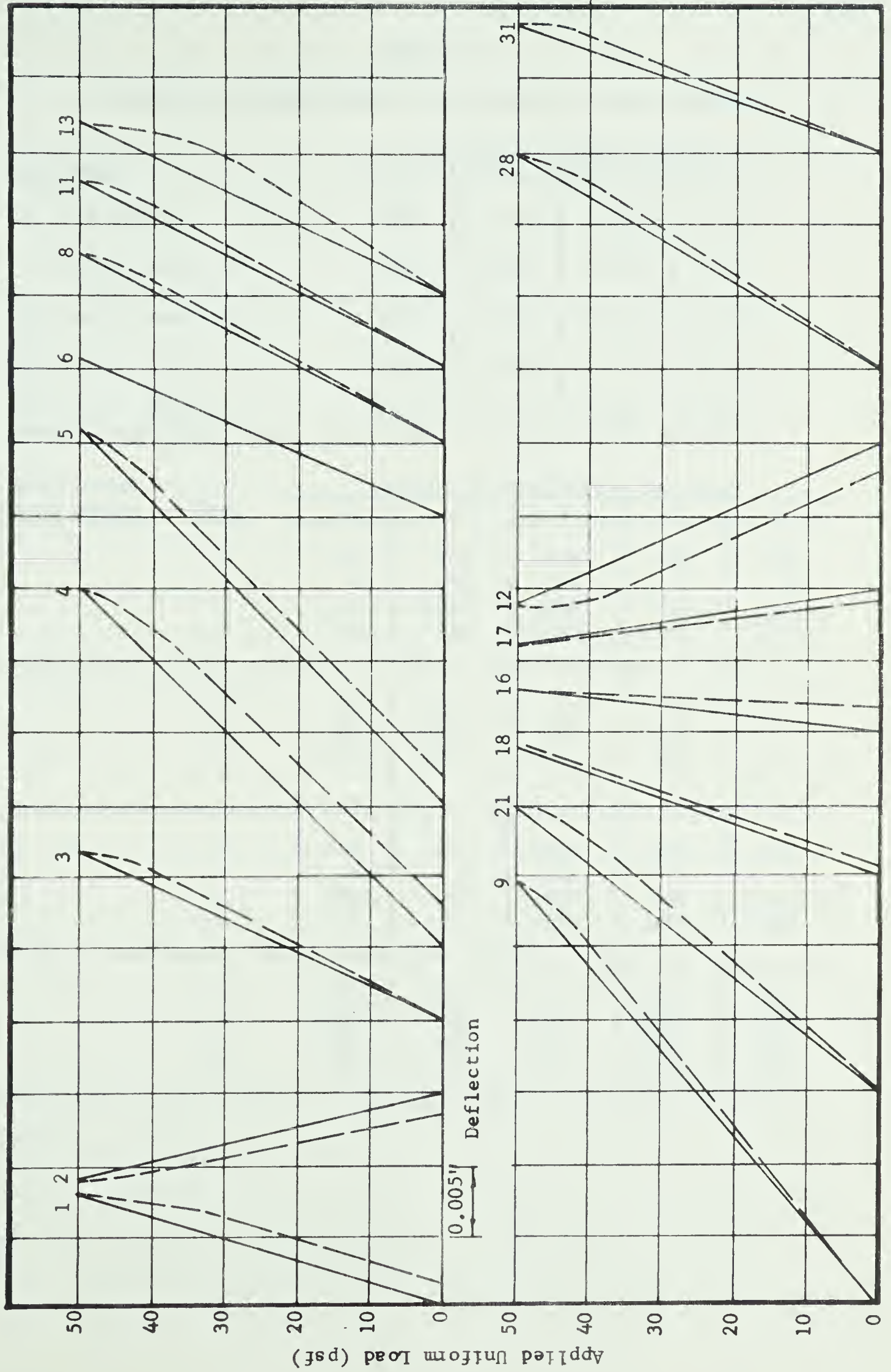


FIGURE 5.8 LOAD-DEFLECTION PLOTS FOR TEST 6

TABLE 5.1

COMPARISON OF DEFLECTIONS FOR VARIOUS LOADING PATTERNS

Test Number	Deflection Gauge Number			
	5	7	15	17
2 All panels	.027"	.029"	.018"	.008"
3 Inner panels NS	-.003"	-.004"	.026"	.023"
4 Inner panels EW	.001"	.027"	.001"	.018"
5 Outer panels NS	.030"	.031"	-.006"	-.009"
6 Outer panels EW	.026"	0	.020"	-.004"

Deflection Gauge Number	Test	Percent of Deflection for Test				
		2	3	4	5	6
5	3	-11	100	-300	-10	-12
	4	4	-33	100	3	4
	5	111	-1000	3000	100	115
	6	96	-870	2600	87	100
7	3	-14	100	-15	-13	--
	4	93	-675	100	87	--
	5	106	-780	114	100	--
	6	--	--	--	--	--
15	3	144	100	2600	-430	130
	4	6	4	100	-17	5
	5	-33	-23	-600	100	-30
	6	111	77	2000	-333	100
17	3	287	100	128	-255	-575
	4	225	78	100	-200	-450
	5	-125	-39	-50	100	225
	6	-50	-17	-22	45	100

the columns (Test 3) results in as much as 28% greater deflection at the center of the interior panel than a strip load of equal magnitude in the perpendicular direction (Test 4). Similar results are observed at the location of deflection gauge number 7 (see FIGURE 3.8), where the application of a strip load on the outer strips parallel to the long faces of the columns (Test 5) resulted in about 14% greater deflections than a strip load in the perpendicular direction (Test 4). Deflection gauge location 5 deflected about 15% more in Test 5 than for an equal loading in Test 6.

These results indicate a relatively stiffer structure in terms of load resistance in directions parallel to the long faces of the column than in the directions perpendicular to this long face. This is undoubtedly a result of shortened effective spans due to the large c/l ratio of the columns.

5.7 Test 7.

Test 7 was a uniform loading of all panels to failure. Load was applied in 10 psf increments up to a maximum load of 250 psf (1DL + 4.3LL) at which time it was felt that the usefulness of the structure had been exceeded due to excessive deflections.

Deflections were measured up to a load of 200 psf at which time the gauges were removed in fear that they would be damaged if left in place longer. As a result, the load-deflection plots presented in FIGURE 5.9 indicate a maximum load of 200 psf. It will be noted that these plots have included the creep of the slab under sustained loads and also the time of application for each load level.

With the exception of points near the center of the structure

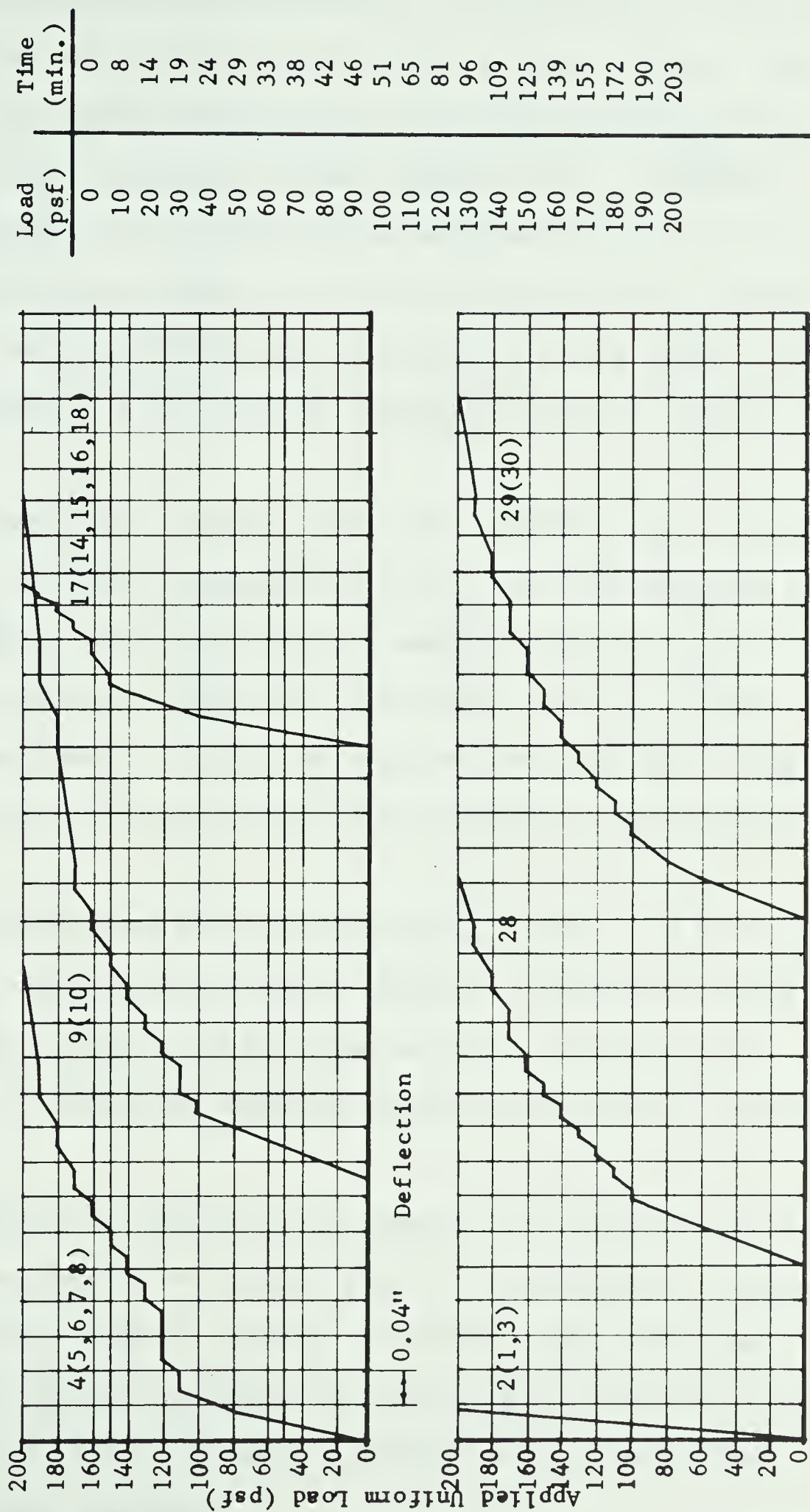


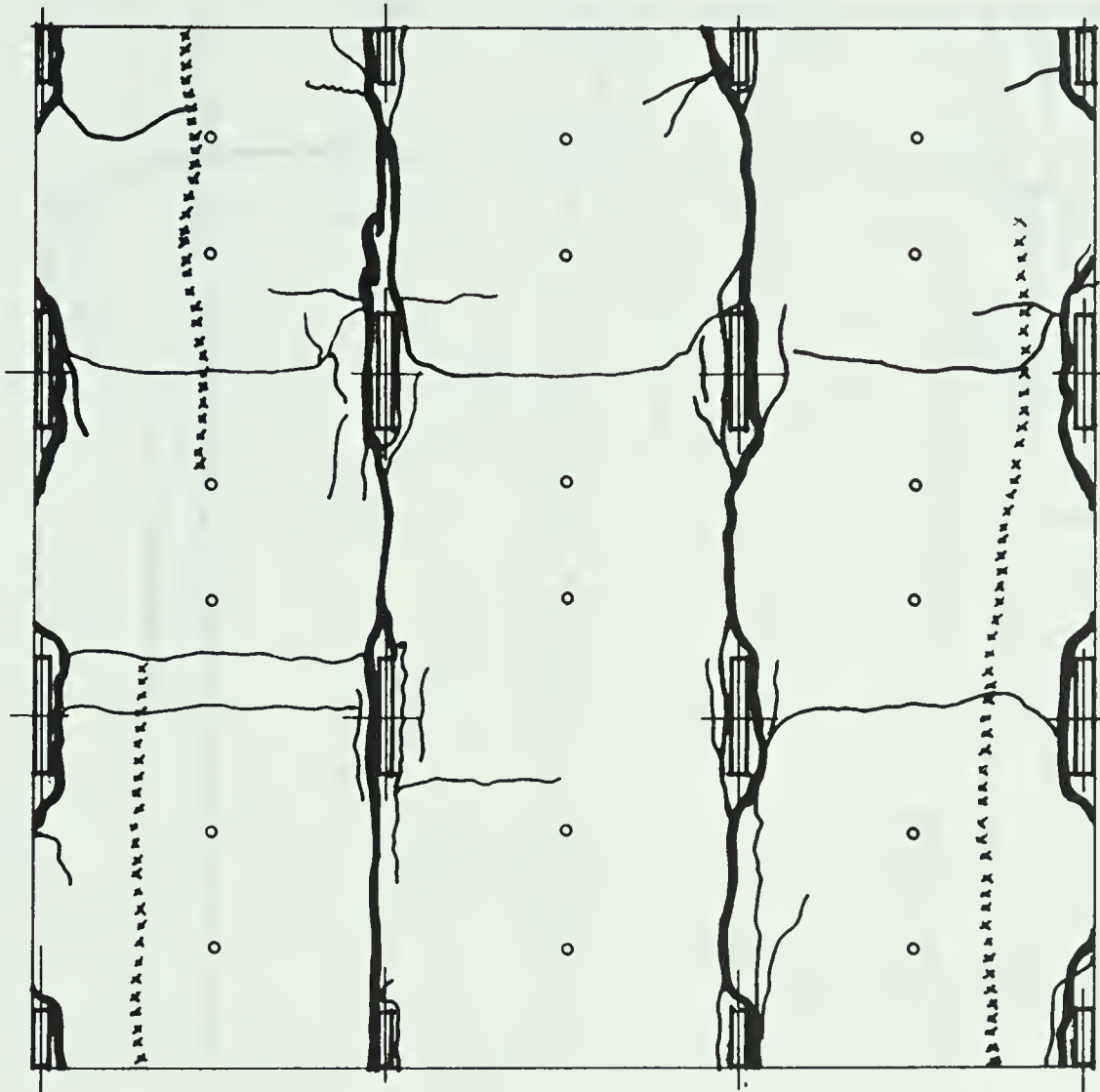
FIGURE 5.9 LOAD-DEFLECTION PLOTS FOR TEST 7

or at the discontinuous edges parallel to the long faces of the columns, the plots indicate increased creep at loads of about 100 psf. The slopes of the loading portions of the plots indicate that there has been some loss in stiffness at load of about 120 psf. Deflection gauge locations near the center of the structure (14, 15, 16, 17 and 18) do not show this behavior until loads of about 150 psf, and the points at the discontinuous edges parallel to the long faces of the columns (points 1, 2 and 3) do not indicate any creep or loss of stiffness.

Assuming that immediate deflections of $L/360$ (0.2") were the governing restriction, excessive deflections were first observed at a load of about 140 psf at deflection gauge locations 9, 10, 29 and 30. This represents a safety factor on design live load of about 2.8, which is satisfactory. Effects of creep and shrinkage over a long period of time would reduce this factor of safety to approximately 1.0.

As can be seen by the crack patterns, FIGURES 5.10 and 5.11, and also the load-deflection plots, FIGURE 5.9, extensive cracking was observed. Cracking did not commence until approximately 80 - 100 psf which is near the maximum load previously applied to the structure.

Cracking was initially most prevalent along the slab top surface on lines parallel to the long faces of the columns and joining the two interior lines of columns. Eventually cracks along these lines became extensive enough to be termed hinges at which time the bottom surface of the slab near the exterior panels' center lines commenced severe cracking.



xxxxxx Indicates crushing on top surface occurring at loads above 240 psf.

FIGURE 5.10 TOP CRACKS AFTER TEST 7



xxxxxx Indicates crushing on bottom surface occurring at loads above 240 psf.

FIGURE 5.11 BOTTOM CRACKS AFTER TEST 7

The slab at the face of the edge columns for the strips parallel to the long face of the columns cracked at loads of 100 to 160 psf. At a load of 220 psf, column 13 completely punched through the slab rendering it inoperative.

Not until loads of about 210 psf was there any cracking on the column center lines perpendicular to the long dimensions of the column. These finally did show up as a result of redistribution of bending moment from the initial behavior of primary one-way action perpendicular to the long faces of the columns.

FIGURE 5.12 shows clearly the one-way action observed in the structure after failure. Areas of most severe damage were the outer panel strips parallel to the long faces of the columns and least damage was observed in the interior panel which was very nearly uncracked. This is reasonable to expect since the membrane forces in a slab tend to counteract the bending moments to a high degree. These forces occur on all four edges of an interior panel, for instance, whereas on edge or corner panels they will be present on only three or two sides respectively.

5.8 Test 8.

Since the center panel was essentially uncracked after Test 7, it was decided to load only this panel in an attempt to get some idea of the action of membrane forces on the interior panels of a slab system and obtain a typical overload factor. Only deflections were measured during this test at deflection gauge locations 12, 16, 17, 18 and 22. These were measured by means of a surveyor's level and five hanging scales mounted on the bottom of the slab. For this reason,

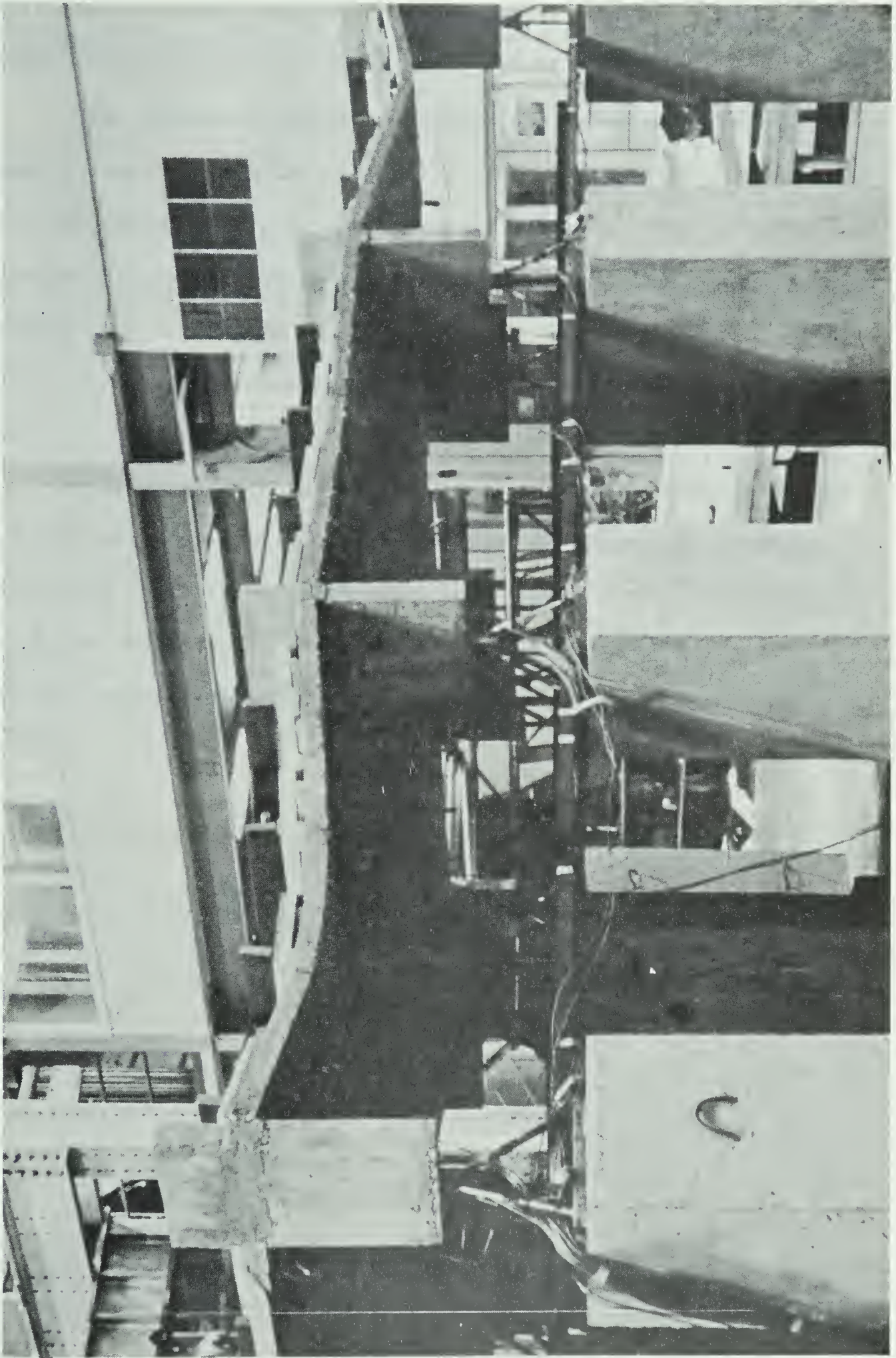


FIGURE 5.12 VIEW OF SLAB SHOWING ONE-WAY ACTION

deflections are only accurate to the nearest $1/16$ of an inch. The results of this test are presented in FIGURE 5.13.

The maximum deflection observed was recorded at deflection gauge location 17 (center of panel). This point reached a deflection of $L/360$ (about $3/16''$) at approximately 205 psf. This represents an overload factor of 1.46 as compared to points 9, 10, 29 and 30 which reached this limitation at about 140 psf. The limitation of $L/180$ (about $13/32''$) was reached at 350 psf which represents an overload factor of 2.0.

The difference in factors is likely due to cracking on the exterior spans of the slab. As the load increased, the cracking approached a hinge condition which resulted in rapid deflections for points on the exterior spans whereas the membrane stresses kept the interior panel from deflecting, thus resulting in a higher overload factor.

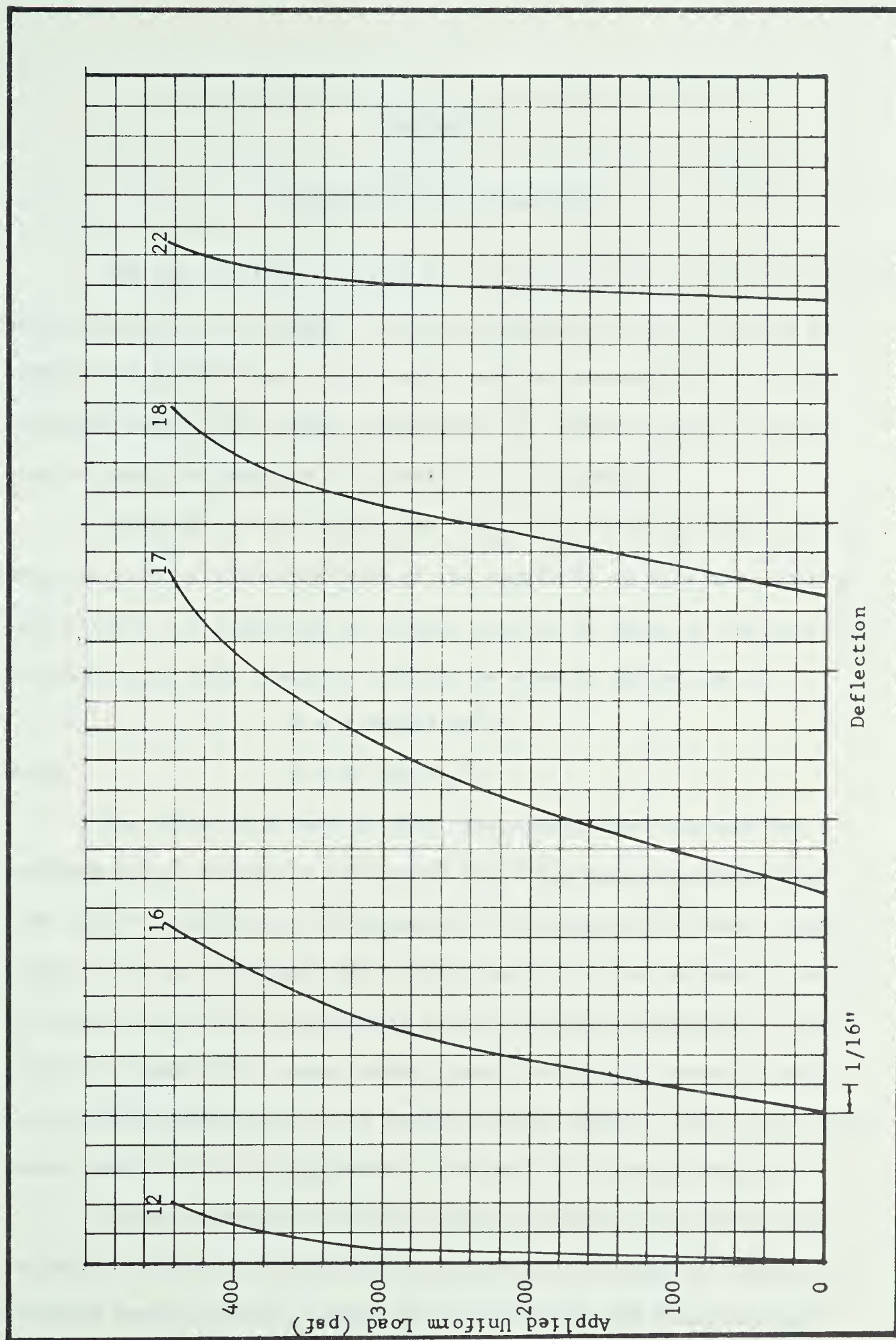


FIGURE 5.13 LOAD-DEFLECTION PLOTS FOR TEST 8

CHAPTER VI

COMPARISON OF DEFLECTIONS

The use of elastic theory for the calculation of deflections for slab structures can consume an enormous amount of time, even for the simplest of structures. As a result, several expressions have been developed empirically which approximate the "exact" value of deflection as predicted by more comprehensive relationships.

Kavanagh, on the other hand, used the method of finite differences to predict the deflection of the center of an interior panel of aspect ratio 1.0 supported on columns similar to those of the test model. The results of this analysis indicate an elastic deflection of

$$\Delta = 0.002739 \text{ } QB^4/D$$

where $D = Et^3/12(1-\mu^2).$

The value of D used in this relationship was computed two separate ways. First the value of E for the concrete as determined from the test cylinders, the nominal slab thickness (2") and a representative value of $\mu(0.20)$ were substituted into the expression for D. The results of these calculations are listed under "Kavanagh - Substituted" in TABLE 6.1. These values appear to compare favorably with the observed deflections up to loads of about 40 psf. Above this load there appears to be a significant difference between the values.

Closer examination of the figures, however, shows that the majority of observed values have a deflection increment of 0.004" for a 10 psf load increment, except in the first two and last two load

TABLE 6.1

COMPARISON OF PREDICTED AND OBSERVED DEFLECTIONS

Load (psf)	Observed Deflection (inches)	Predicted Deflections (inches)			
		Kavanagh		PCA	
		Substituted	Test Beams	Substituted	Test Beams
0	0	0	0	0	0
10	0.002	0.003	0.004	0.002	0.004
20	0.005	0.005	0.007	0.005	0.007
30	0.009	0.008	0.011	0.007	0.011
40	0.013	0.010	0.014	0.009	0.014
50	0.017	0.013	0.018	0.011	0.018
60	0.021	0.016	0.021	0.014	0.021
70	0.025	0.018	0.025	0.016	0.025
80	0.029	0.021	0.029	0.018	0.028
90	0.034	0.023	0.032	0.021	0.032
100	0.043	0.026	0.035	0.023	0.036

increments. The computed deflection increment indicates a deflection increment of about 0.003". Thus the computations agree for the first few loads but begin to deviate significantly as the difference accumulates. These low deflection increments at the beginning of the test could be the result of dial gauge stiffness, or some similar phenomenon, and as such, the first few readings should be ignored and concentration should be given to the mean deflection increment.

The second method used the results of the moment-steel strain beams for calculating the stiffness factor. This was done by solving for EI from known deflections in a beam loaded at the third points. Since the center of the slab is reinforced with 1/8 inch square bars

and the test beam was reinforced with 1/4" bars, the stiffness of the slab had to be extrapolated from the theoretical relationships derived in Chapter IV. These indicate that the deflection should be about 1.42 times as great for the 1/8 inch bars as for the 1/4 inch bars. The results of these computations appear in TABLE 6.1 under "Kavanagh - Test Beams." These values compare very well with regards to the deflection increment, which is slightly less than 0.004" for 10 psf load.

One of the empirical relationships for slab deflections is that published by the Portland Cement Association. This requires the geometric properties of the slab panel plus the modulus of elasticity of the concrete for the solution. In order to apply it to the test structure, however, it was felt that the values substituted into the expression should be modified to consider the effective column reactions concentrated at the column ends. Thus it was assumed that there was a "column" of dimensions equal to the column width (3") and 1.5 times the width (4.5") located at the ends of the real column cross-sections. This results in deflections as indicated under "PCA - Substituted" in TABLE 6.1. These follow an increment slightly less than 0.003" for a 10 psf load increment. However, by using the results of the test beams, the deflection increment becomes slightly less than 0.004", as indicated under "PCA - Test Beams."

The reasons for greater observed deflection increments than predicted is likely the result of deviations of the real structure from the ideal conditions assumed by Kavanagh, plus possible differences in the concrete test cylinders and the slab. This last reason seems likely due to the good agreement observed when the results of

the test beams are used in computation of the stiffness factors.

Thus it seems that in order to accurately predict the deflection of this type of slab system, the stiffness should be determined by tests on representative sections of slab. The results of these tests may then be used reasonably effectively in the modified PCA relationship.

CHAPTER VII

SUMMARY AND CONCLUSIONS

7.1 Summary.

A flat plate floor slab was constructed to a $1/3$ scale and supported on columns measuring 0.40 times the span length in one direction and $1/24$ of the span length in the other direction. The model consisted of nine panels arranged in a three by three grid and so contained two different types of edge panels, four corner panels and one interior panel.

The structure was designed according to the elastic frame analysis method as provided in the American Concrete Institute Building Code (ACI 318-63).

Testing of the structure was carried out under several patterns of simulated uniform loading. Deflections, column reactions, applied loads and slab steel strains were recorded.

Evaluation of bending moments from the slab steel strains were made but due to malfunctioning equipment the steel strain readings were lost, which meant that it was impossible to determine internal stresses in the slab. Similarly the results from the column reactions were lost.

From deflection and load readings and the resulting crack patterns, it was possible to obtain a general behavior of the slab.

7.2 Conclusions.

It was concluded that the overall behavior of such a structure

was one-way slab action perpendicular to the long faces of the columns. This agrees with the predicted behavior from Kavanagh's theoretical analysis for this particular column shape.

The slab behaved satisfactorily at design loads with only minor cracking developing on the top surface on lines joining the interior columns and parallel to the long faces of the columns.

Excessive deflections were reached only at loads of about 2.8 times the design live loads and as such presented no problems. Calculated deflections at the center of the interior panel agreed fairly closely with the theoretical values predicted by Kavanagh with the only problem developing being the determination of the plate stiffness factor, D .

As a result of a test on the interior panel only, an estimate of the overload factor for interior panels in such a structure was found to be about 1.46. This value increased as cracking proceeded in the outer panel strips until it reached 2.0 for deflection limitations of $L/180$ as compared to $L/360$ for the previous value.

Thus, the present method of elastic frame analysis for such a structure yields a safe and satisfactory end result.

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